Learnable evolutionary model for flexible job-shop scheduling

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2007

Ho, N. B. (2007). Learnable evolutionary model for flexible job-shop scheduling. Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/35728>

<https://doi.org/10.32657/10356/35728>

Potential Use of Residual Soil and Geosynthetic Material in Capillary Barrier System

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A thesis submitted to the Nanyang Technological University
in fulfilment of the requirement for the degree of
Doctor of Philosophy

2007

Dedicated to

my parents; my wife, Vivi & my son, Adam

ABSTRACT

Rainfall-induced slope failures frequently occur in residual soil slopes in tropical regions. Previous research works indicated that the main cause of slope failure in residual soil slopes is rainfall infiltration. The infiltrating water decreases matric suction in the soil and consequently reduces shear strength of the residual soil slopes. One of the possible preventive measures for the rainfall-induced slope failures is the use of a soil cover to minimize water infiltration into the slope. Previous research works showed that a capillary barrier system could be used as a soil cover to minimize rainfall infiltration. A capillary barrier is a soil cover consisting of a fine-grained soil layer overlying a coarse-grained soil layer. In this study, the use of a local residual soil derived from Bukit Timah Granite as the fine-grained layer in a capillary barrier system was investigated.

The performance of a capillary barrier system using the residual soil as a fine-grained layer under infiltration and evaporation processes was investigated by means of laboratory model and numerical analyses. The results indicated that the capillary barrier model using residual soil as the fine-grained layer could store the infiltrating water in the fine-grained layer up to its water storage capacity without significant percolation. However, the rate of water storage recovery of the residual soil was very slow. Moreover, after several infiltration and evaporation processes, excessive desiccation cracks occurred in the residual soil layer. Water could easily infiltrate the residual soil layer through the cracks and subsequently percolate into the gravelly sand layer. Therefore, it was proposed to reduce the shrinkage characteristics and improve the hydraulic properties of the residual soil by mixing the residual soil with fine sand at different proportions by dry mass.

Evaporation and shrinkage tests were carried out to obtain the evaporation and shrinkage characteristics of the residual soil and fine sand mixtures. The test results indicated that mixing the residual soil with fine sand could reduce the shrinkage potential and improve the hydraulic properties of the soil.

In addition to the investigation of the residual soil as a fine-grained layer, the use of a synthetic material as a coarse-grained layer in capillary barrier system was also studied. Geosynthetic material was selected due to its characteristics and worldwide availability. The geosynthetic material investigated in this study was Polyfelt Megadrain 2040 manufactured by Polyfelt Geosynthetics. Polyfelt Megadrain 2040 is a drainage product made of a matrix of polypropylene monofilaments sandwiched between two layers of polypropylene geotextile. One-dimensional soil column model and two-dimensional sloping capillary barrier model were constructed to investigate the performance of capillary barrier using geosynthetic material as the coarse-grained layer. The experimental results indicated that the geosynthetic material used in this study performed satisfactorily as a coarse-grained layer in the capillary barrier system. The geosynthetic material provides double protection where the infiltrating water could flow above the geosynthetic material as lateral drainage under unsaturated condition and also inside the geonet drainage layer if percolation through the upper geotextile layer occurred.

ACKNOWLEDGEMENT

Praise be to Allah, Lord of the worlds, who enables me to finish this thesis. I would like to express my great and sincere appreciation to my supervisor, Professor Harianto Rahardjo for his patient guidance, invaluable supervision, and encouragement throughout the research. I am very grateful for the opportunity, supports, suggestions and corrections given by him.

I would like to extend my grateful to Professor D.G. Fredlund from University of Saskatchewan, Canada for the invaluable suggestions given to me.

I also wish to express my gratitude to the technical staff of Geotechnics Laboratory, Mr. Christopher Chia, Mr. Heng Hiang Kim, Mr. Eugene Tan, Mr. Tan Han Guan, Mr. Tay, and Mrs. Lee for their helpful assistance during the experimental works. I also am grateful to my fellow research student in Room b06 for their friendship and generous helps during the study.

I would like to acknowledge the financial support provided by the Nanyang Technological University in the form of a research scholarship. Acknowledgement also goes to Polyfelt Geosynthetics who provided the Polyfelt Megadrain 2040 for this study.

Last but not least, I am grateful to my wife, Vivi and my parents for their support during the study.

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Chapter 1

INTRODUCTION

1.1 Background

Slope failures frequently occur in residual soil slopes in tropical regions. The main cause of slope failure in residual soil slopes is rainfall infiltration (Brand, 1984). Previous research works indicated that infiltration has a significant effect in reducing matric suction and consequently shear strength of residual soil slopes (Rahardjo et al., 2005, Tsaparas et al., 2001; Zhang et. al., 2000; Gasmo et al., 2000; Rahardjo et al., 1995). One of the possible preventive measures for the rainfall-induced slope failure is by minimizing water infiltration into the soil slopes. Previous research works indicated that capillary barrier system could be used as a soil cover system to minimize rainfall infiltration (Tami et al., 2004b; Khire et al., 2000; Morris and Stormont, 1997; Stormont, 1997).

Availability of materials is one of the important considerations in the design of capillary barrier system. Materials which are not available near the site could be costly to be used in the capillary barrier system. Therefore, the possibility of using local soils as a capillary barrier material needs to be investigated. In this study, the application of residual soil as a fine-grained layer in a capillary barrier system was investigated. A residual soil derived from Bukit Timah Granite was used in this study. As described by Poh et al. (1985), the residual soil from Bukit Timah Granite is commonly a sandy clayey silt.

Previous research works indicated that a capillary barrier constructed using a clayey material did not exhibit good performance due to the extensive desiccation cracking (Tami, 2004; Khire et al., 2000; Montgomery and Parsons, 1990). Moreover, the recovery of water storage in a clayey soil was very slow as observed in a laboratory capillary barrier model (Tami, 2004). In this study, the residual soil was mixed with fine sand at different proportions by dry mass to improve its hydraulic properties and volume change characteristics. It is desirable to obtain a

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mixture that has a low volume change and a speedy recovery of water storage during a drying process. The effect of evaporation on the recovery of water storage of the residual soil was also investigated.

The use of a synthetic material as a coarse-grained layer in capillary barrier system was also studied. Geosynthetic material was selected due to its characteristics and worldwide availability. Although the geosynthetic capillary barrier system has been studied in the pavement design (Stormont, 1998; Henry and Holtz, 2001), its application for slope stabilization has never been fully investigated.

1.2 Objectives

The main objective of this research is to investigate the performance of capillary barriers constructed using locally available materials. A local residual soil derived from Bukit Timah Granite and a mixture of the residual soil and fine sand were used as the fine-grained layers in capillary barrier models. The use of geosynthetic material (Polyfelt Megadrain 2040) as the coarse-grained layer in a capillary barrier system was also investigated. The mechanism of capillary barrier constructed using the geosynthetic material as a coarse-grained layer was studied. The performance of the capillary barriers under different infiltration and evaporation rates were also investigated.

1.3 Methodology and Scope of Research

This research was focused in studying the performance of sloping capillary barriers constructed using a local residual soil as the fine-grained layer. Capillary barrier system with a steep slope angle of 30° was investigated for its potential use as a slope stabilization method against rainfall-induced slope failure. The use of a geosynthetic material as a coarse-grained layer was also investigated in this study.

This research consisted of mainly laboratory experiments. Nevertheless, numerical analyses were also carried out. The performance of capillary barrier system under infiltration and evaporation processes was studied using a soil column apparatus and an infiltration box apparatus. The soil column apparatus was used to study the mechanism of capillary barrier system constructed using a geosynthetic material as

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the coarse-grained layer. The infiltration box apparatus was used to investigate sloping capillary barrier model under different infiltration and evaporation processes. Three different materials were used as the fine-grained layer, i.e. fine sand, residual soil and the mixture of residual soil and fine sand. Gravelly sand and geosynthetic material were used as the coarse-grained layer in the capillary barrier models.

Numerical analyses were conducted using the saturated/unsaturated finite element seepage modeling software, SVFlux (Soilvision System Ltd., 2005). The results of numerical analysis were compared with the experimental data to verify the numerical model adopted in the study. Parametric studies to investigate several parameters and conditions were then conducted using the verified numerical model

1.4 Outline of the Thesis

This thesis is organized in the following manner:

Chapter 1 describes the background, objective, and methodology and scope of the research.

Chapter 2 presents the literature review related to residual soils, rainfall-induced slope failures, capillary barrier system, and also infiltration and evaporation processes in a soil layer.

Chapter 3 explains the theory of unsaturated soil mechanics related to this study.

Chapter 4 describes the research programme, including the laboratory experiments, numerical analyses, and parametric study. The equipments used in the laboratory study are also described in this chapter.

Chapter 5 presents the results obtained from the laboratory experiments, numerical analyses, and parametric study.

Chapter 6 discusses the results presented in Chapter 5.

Chapter 7 presents the conclusions of the study and also recommendations for future study.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

This chapter reviews some literature related to this research. The overview of residual soils and the residual soil of Singapore is presented in Section 2.2. The subsequent section presents an overview on rainfall-induced slope failures. In Section 2.4, the mechanism of capillary barrier and the development of the study on capillary barrier system are discussed. Geocomposite capillary barrier system as one type of capillary barriers is also discussed in this section. Flux boundary conditions on the capillary barrier system are discussed on Section 2.5. Finally, a brief review on the shrinkage and cracking behaviour of soil is presented in Section 2.6.

2.2 Residual Soil

This section provides an overview of general characteristics of residual soil and the characteristics of residual soil in Singapore.

2.2.1 General

Residual soils are formed by insitu weathering of igneous, sedimentary, metamorphic or volcanic rocks. The main agencies of weathering are physical, chemical and biological processes. Blight (1997) defined residual soil as soil-like material derived from the in situ weathering and decomposition of rock which has not been transported laterally from its original location. A typical tropical residual soil profile is shown in Figure 2.1 (Little, 1969; after Blight, 1997).

There are many classification systems of residual soil, which are commonly based on degree of weathering and engineering properties. The modified weathering classification for the granitic and the sedimentary residual soil as adopted by

Dames and Moore (1983) is recommended by Winn et al. (2001) because it is easy to differentiate the grades on small volumes of samples.

Figure 2.1 Typical tropical residual soil profile (Little, 1969; after Blight, 1997)

Residual soils commonly exist above the groundwater table, which means that they are in an unsaturated condition. Fredlund and Rahardjo (1993a) pointed out that the primary factor contributing to the unusual behaviour of residual soils are their negative pore-water pressure.

Residual soils can be found in many parts of the world, especially in the tropical regions such as northern Brazil, Ghana, Nigeria, southern India, Sri Lanka, southern China, Thailand, Burma, Philippines, Indonesia, Malaysia, and Singapore.

2.2.2 Residual soils of Singapore

Almost two-thirds of Singapore is covered by residual soils from the Sedimentary Jurong Formation and the Bukit Timah Granite Formation. Each of these two residual soils covers approximately one-third of Singapore Island. The residual soil from the Sedimentary Jurong Formation can be found on the western part of Singapore and the residual soil derived from the Bukit Timah Granite occupies the central and northern parts of Singapore as shown in Figure 2.2.

Figure 2.2 Generalized geological map of Singapore (Public Works Department, 1976)

The residual soils of Singapore are often spatially heterogeneous with their consistency or hardness often varying significantly, especially throughout the depth of their formations (Chang, 1988). The variability of the residual soils can be observed from its large band of grain size distributions as shown in Figure 2.3. Based on plasticity chart, Poh et al. (1985), Winn et al. (2001), Leong et al. (2003), concluded that the sedimentary residual soil consisted of clayey materials and the granitic residual soil consisted of silty materials. Poh et al. (1985) described that the residual soil from the Bukit Timah Granite varied from silty or clayey sands to silty or sandy clays (depending on the degree of weathering) but commonly was sandy clayey silt.

(a) Bukit Timah Granitic residual soils

(b) Jurong Sedimentary residual soils

Figure 2.3 Grain size distribution envelopes of residual soils (Leong et al., 2003)

Similar to the variability of its grain size distribution, the unsaturated soil properties of the residual soils, such as soil-water characteristic curves (SWCC) are also highly variable. Agus et al. (2001) presented the SWCCs of the residual soils from Sedimentary Jurong Formation and the Bukit Timah Granite (Figure 2.4). SWCC tests were carried out on samples obtained from various depths to

investigate the effect of weathering on the shape of the SWCC. The results indicated that the depth of weathering did not have a consistent effect on the SWCCs. The parent rock type has more significant effect on the SWCCs.

(a) Bukit Timah Granitic residual soils

(b) Jurong Sedimentary residual soils

Figure 2.4 Soil-water characteristic envelopes of residual soils (Agus et al., 2001)

2.3 Rainfall-induced Slope Failure

Slope failures in residual soils can be attributed to several factors, such as geological activities, hydrological influence, topographical, weathering, human interference and climatic conditions (Brand, 1984; Broms and Wong, 1991; Sower, 1992). However, rainfall is believed to be the major cause of landslide in residual soils (Brand, 1984).

Residual soil slopes commonly have a deep groundwater table. The soil above the groundwater table is unsaturated with negative pore-water pressure (Rahardjo et al., 1995). The negative pore-water pressure provides an additional shear strength to unsaturated soils. However, this additional shear strength will decrease or even disappear as rainwater infiltrates the unsaturated soils. As water infiltrates the soil, the pore-water pressure in the soil increased (matric suction decreased). A reduction of matric suction in unsaturated soils is equivalent to a decrease in the effective stress acting on the soils. This will cause the slope to be more susceptible to failure.

Recent studies on rainfall-induced slope failures (e.g., Rahardjo et al., 2005; Tsaparas et al., 2002; Zhang et al., 2000; Gasmo et al., 2000; Toll et al., 1999; Lim et al., 1996; Rahardjo et al., 1995) indicated that rainfall infiltration has a significant effect in reducing the matric suction and consequently the shear strength of residual soil in Singapore. Based on the observation of past slope failures in NTU, Rahardjo et al. (2001) concluded that a five day antecedent rainfall exceeding 60 mm together with a daily rainfall greater than 90 mm appeared to have been sufficient to cause landslides. The study concluded that both daily and antecedent rainfalls are important factors for the occurrence of landslides.

The hydrologic responses of residual soil slopes in Singapore due to rainfall events were investigated by Rezaei et al. (2003). Three instrumented hillslopes in Singapore were monitored for a period of 420 days in order to observe the impact of rainfall on the pore-water pressure changes and runoff generation. The results showed that a threshold rainfall of about 10 mm is required to produce run-off. The relationship between the increase in pore-water pressures and daily rainfall amount was provided. However, it should be noted that the use of the relationship

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should be restricted to similar field conditions under which the measurement took place. Rahardjo et al. (2005) also investigated the response of residual soil slope due to rainfall. A residual soil slope was instrumented for the measurement of runoff, pore-water pressure, and water content changes. Natural and simulated rainfalls (total rainfall amount from 0.5 mm to 150 mm) were applied to the instrumented slope. The results indicated that a large proportion of the rainfall contributes to infiltration in the residual soil slope. A relationship between the amount of infiltration and total rainfall amount was presented. The amount of rainfall which was converted into infiltration was from 40% to 100% of total rainfall, depending on the rainfall amount. One hundred percent (100%) of the rainfall infiltrated into the slope when the total rainfall amount was less than 10 mm. The percentage of infiltration decreased gradually from a total rainfall of 10 mm up to a total rainfall of 100 mm. When the total rainfall was higher than 100 mm, the percentage of rainfall which was converted into infiltration was constant at 40%.

Other than field experimental studies, several researchers also investigated rainfall-induced slope failures based on analytical analysis. Collins and Znidarcic (2004) studied the effect of both negative and positive pore-water pressure on the stability of initially unsaturated soil slopes. The study was based on one-dimensional flow modeling and infinite slope analysis method. The results showed that in general, shallow slope failures are associated with the development of positive pore-water pressure, while deeper slope failures are associated with a loss in suction. Other analytical study by Zhan and Ng (2004) was carried out based on the analytical solution by Srivastava and Yeh (1991). Several parameters include saturated permeability (k_s), desaturation coefficient (α), water storage capacity ($\theta_s - \theta_r$), and antecedent and subsequent rainfall infiltration rates (q_A and q_B) were considered in this analytical study. The results indicated that infiltration process and pore-water pressure changes are primarily controlled by both $q\alpha/k_s$ and k_s/α . In general, the larger the value of $q\alpha/k_s$, the greater the reduction of negative pore-water pressure in the shallow layer and the larger the value of k_s/α , the faster the advancement of wetting front.

2.4 Capillary Barrier

This section explains the mechanism, materials and development of capillary barrier system. Review of literature on the capillary barrier system using geosynthetic material is also presented in this section.

2.4.1 The mechanism of capillary barrier

A capillary barrier is a soil cover employing a fine-grained soil layer overlying a coarse-grained soil layer. This configuration of soil cover is called capillary barrier in reference to the mechanism of capillary tension that limits water's downward movement from a finer soil into a coarser soil (Stormont and Anderson, 1999). The basic form of a capillary barrier system is shown in Figure 2.5.

Figure 2.5 Basic design of a capillary barrier system

The barrier effect in a capillary barrier system can be explained using the soil-water characteristic curves and the permeability functions of the capillary barrier materials. A soil-water characteristic curve (SWCC) is defined as a relationship between volumetric water content, θ_w and matric suction, $(u_a - u_w)$, while a permeability function is defined as a relationship between the coefficient of permeability, k_w , and matric suction, $(u_a - u_w)$. Figures 2.6 and 2.7 show the typical

soil-water characteristic curves and permeability functions of capillary barrier materials.

Figure 2.6 Soil-water characteristic curves of capillary barrier materials

Figure 2.7 Permeability functions of capillary barrier materials

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As can be seen from Figures 2.6 and 2.7, the shape of permeability function is similar with the shape of SWCC. This is due to the fact that the coefficient of permeability of an unsaturated soil is related to the volumetric water content of the soil. In fact, the permeability function can be generated based on the SWCC and the saturated coefficient of permeability, k_s . The SWCC and permeability function are explained further in Chapter 3.

The barrier effect that limits downward water movement in a capillary barrier system is caused by the contrast in the unsaturated coefficient of permeability between the fine-grained and coarse-grained soil layers. The mechanism of capillary barrier can be explained by considering Figure 2.7. At high matric suction values, the coefficient of permeability of the coarse-grained layer is much lower than that of the fine-grained layer. Under this condition, water will not flow into the coarse-grained layer. Water will be stored or diverted laterally through the fine-grained layer. As the infiltration process continues, the volumetric water content of the fine-grained layer increases, the coefficient of permeability of the fine-grained layer also increases. When the infiltrating water reaches the fine-coarse interface, the matric suction of the coarse-grained layer will decrease. When the matric suction in the coarse-grained layer reaches its water-entry value, ψ_w , (Figure 2.6) the coefficient of permeability of the coarse-grained layer will increase. Water will start to flow into the coarse-grained layer. As the matric suction decreases further, the coefficient of permeability of the coarse-grained layer will increase rapidly and finally exceeds that of the fine-grained layer. At this condition, there will be no barrier effect from the coarse-grained layer. Water will flow easily into the coarse-grained layer and capillary barrier is no longer effective in preventing downward water movement.

It can be concluded that capillary barrier is effective as long as the matric suction at the interface is higher than the water-entry value of the coarse-grained layer. Previous data by Stormont and Anderson (1999) showed that breakthrough into the coarse-grained layer will only occur after the matric suction value at the surface of coarse-grained layer decreased to its water-entry value.

2.4.2 Materials of capillary barrier

Materials of capillary barrier system, including the fine-grained and coarse-grained layers are described in this section.

2.4.2.1 Fine-grained layer

The fine-grained layer in capillary barriers acts as a temporary water storage. Hence, the thickness of the fine-grained layer is an important design variable for capillary barrier. Simulation results by Khire et al. (2000) indicated that thickness and hydraulic properties of the fine-grained layer have significant effects to the water balance of capillary barriers. The field observation indicated that increasing the thickness of the fine-grained layer will reduce percolation because the water storage capacity becomes larger. Similar results were also obtained by Tami (2004). Based on the parametric studies using a numerical model which was already verified by the experimental data, Tami (2004) concluded that the amount of breakthrough decreased when the thickness of the fine-grained layer of a capillary barrier increased.

Coefficient of permeability of surface layer affects the performance of capillary barriers. Surface layer with a low saturated coefficient of permeability will generate more runoff, less infiltration into the surface layer, and less percolation. Simulation results from Khire et al. (2000) showed that capillary barriers with lean clay (CL) as a surface layer will generate less percolation. Soils that are selected as a surface layer should have resistance to desiccation cracking and suitable for vegetation. A field trial of a capillary barrier by Montgomery and Parsons (1990) illustrated extensive desiccation cracking occurred in the surface layer which was formed by lean clay. Hence, lean clay is not suitable to be used as a fine-grained layer in a capillary barrier system. In a laboratory experiment of sloping capillary barriers, Tami (2004) also observed an excessive desiccation cracking on a capillary barrier model constructed using clayey sand as the fine-grained layer. Tami (2004) suggested that a saturated coefficient of permeability of 10^{-5} m/s can be used as a threshold value since a soil with a saturated coefficient of permeability lower than

10^{-6} m/s is considered as poor drainage soil (Casagrande 1938, after Holtz and Kovacs, 1981).

2.4.2.2 Coarse-grained layer

Unlike the fine-grained layer, the thickness of the coarse-grained layer does not affect much the performance of capillary barriers. Morris and Stormont (1997) pointed out that the coarse-grained layer thickness does not influence the capillary barrier performance as long as it is greater than the minimum required to create the capillary break. Simulation results from Khire (1995) showed that the difference in percolation from three different thicknesses of coarse-grained layer (30 cm, 45 cm, and 75 cm) were less than 0.1 cm. Based on the parametric study, Tami (2004) also concluded that the thickness of the coarse-grained layer has a small impact on the performance of a sloping capillary barrier.

Baker and Hillel (1990) concluded that the coarser and the more uniform a soil texture is, the lower the breakthrough head is expected to be. Stormont and Anderson (1999) proved that this conclusion is plausible. Infiltration tests on soil columns of silty sand over pea gravel, concrete sand over pea gravel, and silty sand over concrete sand were studied. The tests were carried out in order to investigate the capillary barrier effect of the underlying coarser soil. The result showed that breakthrough occurred at a greater suction head when the coarser layer was concrete sand instead of pea gravel. This result indicates that the coarser and the more uniform the coarse-grained layer is, the more effective the capillary barrier will be. Smesrud and Selker (2001) suggested that 80% of the maximum diversion can be obtained with an underlying coarser material that is only 2.5 times coarser than the overlying fine material (calculated by D_{50} of the soil).

Recent study by Tami (2004) indicated that the important parameter of the coarse-grained layer is its water-entry value. As discussed in Section 2.4.1, water will start to flow into the coarse-grained layer if the matric suction in the coarse-grained layer reaches its water-entry value, ψ_w . Thus, it is preferred to have a coarse-grained layer that has a low water-entry value (preferably less than 1 kPa).

2.4.3 Previous research works on capillary barrier system

This section provides summaries on the previous research works on capillary barrier system. The summaries are classified into three methods of studies, which include numerical simulations, laboratory experiments, and field tests.

2.4.3.1 Numerical simulations

Numerical simulation has been used widely in the analysis of capillary barriers. Oldenburg and Pruess (1993) reported that the early numerical study by Frind et al. (1977) considered only the fine-grained layer of a capillary barrier. The assumption made on the numerical analysis was that breakthrough would occur and the capillary barrier would fail if the pressure head at the fine-coarse interface became greater or equal to zero. Other numerical simulation by Johnson et al. (1983) considered both fine-grained and coarse-grained layers. It was found that breakthrough could occur even when the pressure head at the fine-coarse interface was still negative (Oldenburg and Pruess, 1993).

Oldenburg and Pruess (1993) conducted a numerical simulation to represent flow exclusion and leakage effects at capillary barrier using several finite difference methods. The results of the numerical simulation of capillary diversion in an idealized system showed a good agreement with the theoretical analysis from Ross (1990). Thus, the numerical analysis also validates the utilization of quasi-linear theory in describing the permeability functions. Unlike the quasi-linear model of Ross (1990), the numerical simulation also showed the development of breakthrough process. It was reported that the breakthrough region is characterized by a large downward flux through the fine-coarse interface.

Morris and Stormont (1997) investigated the efficiency of capillary barriers as compared to conventional covers based on the amount of moisture that flows through the cover system over a modelling period. Two computer programs were used to evaluate the performance of the capillary barriers and the conventional soil covers. The HELP (Hydrologic Evaluation of Landfill Performance) program was used to obtain the site and design specific flux variables (infiltration and evapotranspiration). The output of the HELP program were used as sources in

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TRACER3D (Travis and Birdsell, 1991), a code that is capable of solving transient two-phase flow. Percolation through the modeled cover system at a site for a given climatic condition was provided by the TRACER3D program. Thus, the performance of the capillary barriers and the conventional covers could be compared directly. Three types of soil covers were modeled in this study, which are:

- Minimal subtitle D cover, a 0.15 m vegetative layer over a 0.45 m infiltration layer, which has a saturated hydraulic conductivity of 10^{-7} m/s or less.
- Standard capillary barrier cover consists of 0.6 m of vegetative soil overlying a 0.3 m gravel layer.
- Capillary barrier with an additional 0.2 m of sand as a transport layer, placed in between the vegetative and the gravel layer.

The simulations were carried out at five different sites in the United States (Albuquerque, Chicago, Columbia, Salt Lake City, and San Francisco) using 10 years of climatic data. The simulation results showed that in four of the five cases, the performance of capillary barriers was equal or better than the minimal subtitle D covers. The capillary barrier in Columbia is less effective due to the high precipitation over the modelling period. The study also found that when the transport layer was added into the capillary barrier system, percolation could be reduced greatly.

Further numerical simulations by Morris and Stormont (1998) showed that water movement within a capillary barrier system could be simulated using TRACER3D finite difference code. Two different models of capillary barrier (with and without transport layer, Figure 2.8) were simulated. The results showed that the simulation using the wetting properties appeared to fit the field data as infiltration was initiated. When the infiltration was stopped and the soil moisture content decreased, the drying properties appeared to be a better representation of field condition. It was suggested that a combination of wetting and drying properties should be used to obtain an accurate simulation.

Figure 2.8 Profile of homogeneous and layered capillary barrier (Morris and Stormont, 1998)

A similar finding was also presented by Tami (2004). Numerical analyses of sloping capillary barriers employing both drying and wetting hydraulic properties were conducted using the SVFlux software (Soilvision Ltd., 2001). The results of the numerical analyses showed that the hysteretic behaviour of the soils should be considered in the numerical simulations to obtain realistic results.

Further study on the transport layer (also called as unsaturated drainage layer) was conducted by Morris and Stormont (1999). TRACER3D software was used to analyze the performance of capillary barrier with an unsaturated drainage layer (UDL). The surface layer was 0.66 m of a finer, uncompacted soil, and the coarser layer was medium gravel. Three materials were used as the UDL: (1) a 100-mesh, uniform sand; (2) a concrete sand that has a less uniform grain size; and (3) a Plainfield sand. The UDL was 200 mm in thickness and was placed between the fine-grained and coarse-grained layers in the capillary barrier system. The numerical simulations were conducted on capillary barriers with 5, 10, and 20% slope. The models were subjected to three different precipitation patterns from three sites in the United States: San Francisco, Chicago, and Columbia, S.C. The results showed that the inclusion of UDL will significantly improve the performance of capillary barriers. Percolation during periods of high infiltration and low evapotranspiration could be avoided due to a large quantity of water removed from the finer layer by lateral diversion. Thus, capillary barriers can be utilized in a wetter climate because reliance on evapotranspiration to remove

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moisture is reduced. Morris and Stormont (1999) also pointed out that the Plainfield sand performed better than 100-mesh sand and concrete sand because it has a higher conductivity over a larger soil suction range. Another important conclusion was regarding the diversion length of the capillary barriers. The diversion length of a capillary barrier is directly related to the slope of the fine-coarse interface. If the interface slope is doubled, the diversion length will also double.

The feasibility of using capillary barrier for slope stabilisation purpose in tropical regions was also studied by Ong (2002) using numerical analyses. Several design parameters of capillary barrier (i.e., combination of soil types, proportion of fine-grained layer and coarse-grained layer, thickness of capillary barrier and extension length of capillary barrier) were examined in the study. It was concluded that selection of soils for capillary barrier is the most important aspect in designing a capillary barrier. It was recommended that the coarse-grained layer in a capillary barrier needs to have a low water-entry value, ψ_w (preferably $\psi_w \leq 1$ kPa) and a steep permeability function. For the fine-grained layer, it was recommended to use a soil with a low saturated coefficient of permeability. However, problems associated with desiccation cracking of the fine-grained soil need to be examined first before using it in a capillary barrier.

Bussiere et al. (2003) conducted parametric studies to study the performance of capillary barrier system for gas migration barriers. The parametric studies were performed based on numerical model which was verified using laboratory experiment and field investigation. It was indicated that the geometry of the capillary barrier system influence its hydraulic behaviour, and therefore, its ability to reduce gas migration. Recent numerical analyses of an inclined capillary barrier soil cover installed on a waste rock piles was conducted by Aubertin et al. (2006). The purpose of the study was to investigate the short-term response of the cover following major storms, for different precipitation rates, material properties, and cover configurations. The results indicated that an inclined capillary barrier system could effectively divert water along the interface of fine-grained and coarse-grained soil layers. The diversion capacity of the capillary barrier system was

affected by the precipitation rate, slope angle, and also the saturated coefficient of permeability of the materials.

2.4.3.2 Laboratory experiments

Several laboratory experiments were conducted to observe lateral diversion flow above the coarse-grained layer in a capillary barrier system. Miyazaki (1988, 1990) investigated the effects of very coarse materials on down-slope water flow using an inclined soil container (160 cm long, 20 cm wide, and 150 cm deep) placed under a rainfall simulator. The laboratory model consisted of sandy loam as the fine-grained layer and gravel as the coarse-grained layer. Experiments by Miyazaki showed that the water flow from the fine-grained to the coarse-grained layer was diverted laterally along the fine-coarse interface (Figure 2.9 a).

(a) Water flow in the inclined fine-coarse interface (Miyazaki, 1993) (b) Vertical distribution of the discharge in capillary barriers (Steinert, 1997; after Kämpf et al., 1998)

Figure 2.9 Lateral diversion in the capillary barrier systems

Steinert et al. (1997; after Kämpf et al., 1998) used an 8 m long flume (Figure 2.10) to model a large scale field capillary barrier system on the landfill "Monte Scherbelino", Frankfurt. Laboratory tests were performed to select the appropriate material and inspect the flow patterns in capillary barriers. The inclination of the flume was set to 10° according to the situation on the field test. The flume experiments represented the downslope part of a capillary barrier system, in which

a considerable amount of water had infiltrated through the upper part and had been diverted laterally above the fine-coarse interface. Tracer injections showed the expected vertical distribution of discharge within the layer (Figure 2.9 b).

Figure 2.10 Flume experiments for the inspection of the flow process in capillary barrier (Steinert, 1997; after Kämpf, 1998)

Tidwell et al. (2003) conducted a flow visualization experiment to investigate the capillary barrier performance as engineered backfill system to divert water away from radioactive waste. The experimental system was a 140 cm diameter and 2.5 cm thickness cell equipped with tensiometers as shown in Figure 2.11. Two different experiments were conducted simultaneously. On the left side of the chamber, the capillary barrier was drained by a simulated fracture system using wicks. The wicks supplied suction to the chamber edge, limited by their length, as might be expected of a fracture network with similar fracture spacing. On the right side of the cell, no wicks were installed, so water will only flow through a single gravity drain located at the bottom of the cell. The experimental results showed that in the early wetting process, the water flow on the left side was almost entirely captured by the wicks, while the water flow on the right side was conveyed to the bottom of the cell by gravity-driven fingers formed along the cell edge. Based on the photographic, outflow, and water pressure data, Tidwell et al. (2002) concluded that water flows on both sides of the cell moved primarily along the capillary

barrier interface in the fine sand layer to the edges of the cell. It was suggested that some means of draining water out from a capillary barrier system is necessary.

Figure 2.11 Schematic of experiment test cell (Tidwell et al., 2002)

Tami (2004) investigated the potential use of capillary barrier system as a slope stabilization technique by limiting infiltration into a soil slope. An infiltration box of 2.45 m in length, 2.00 m in height, and 0.40 m in width was used to construct the laboratory capillary barrier model. Three combinations of materials consisted of 0.2 m thick fine sand overlying 0.2 m thick gravelly sand, 0.4 m thick fine sand overlying 0.2 m thick gravelly sand, and 0.2 m thick silty sand overlying 0.2 m thick gravelly sand were investigated. The inclination angle of the capillary barrier models were 30°. From the results, it was concluded that capillary barriers have high potential application for the slope stabilization measure against rainfall-induced slope failure.

Other than two-dimensional laboratory capillary barrier experiments, several researchers also investigated capillary barrier in one-dimensional soil columns. Stormont and Anderson (1999) constructed three soil columns consisted of silty sand over pea gravel, concrete sand over pea gravel, and silty sand over concrete sand. The soil columns were tested to investigate the capillary barrier effect on an

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underlying coarse-grained soil layer. The results showed that breakthrough into the coarse-grained layer occurred when the suction head at the interface reached the water-entry value of the coarse-grained layer, regardless of the infiltration rate or the properties of the overlying fine-grained soil layer. The test using concrete sand as the coarse-grained layer showed that breakthrough occurred at a much higher suction head than the test using pea gravel. Thus, it was concluded that capillary barrier is more effective if the lower soil layer is more uniform and coarser. In other words, a soil with a low water-entry value would be effective as a capillary break layer.

Similar infiltration tests on soil columns were also conducted by Yang (2002). The infiltration tests were conducted using a 1.0 meter high and 0.19 m diameter acrylic column equipped with Time Domain Reflectometry and Tensiometer for measurements of volumetric water content and negative pore-water pressure, respectively. The infiltration tests were conducted using several combinations of capillary barrier material, such as fine sand overlying medium sand, medium sand overlying gravelly sand, fine sand overlying gravelly sand, clayey sand overlying medium sand, silty sand overlying fine sand, and silty sand overlying gravelly sand. It was concluded that a saturated coefficient of permeability ratio of 2 to 3 orders of magnitude between the fine-grained and the coarse-grained layers was generally effective to produce a capillary barrier effect. Yang (2002) also suggested that the minimum thickness of the coarse-grained layer required in a capillary barrier system is approximately equal to the residual matric suction head of the coarse-grained layer, which is also close to the water-entry value of the coarse-grained layer. A coarse-grained layer which is thicker than the minimum thickness may not affect the barrier effect, while a coarse-grained layer which is thinner than the minimum thickness will reduce the barrier effect.

Using similar soil column apparatus, Indrawan (2005) investigated the use of soil mixtures as fine-grained layers in capillary barriers. The soil mixtures used as the fine-grained layers were a soil mixture of 50% residual soil and 50% gravelly sand (RG-50), a soil mixture of 75% residual soil and 25% medium sand (RM-75), and a soil mixture of 95% residual soil and 5% lime (RL-95). The results of the study

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indicated that the RG-50 and RM-75 were more effective than the RL-95 in maintaining matric suctions during wetting processes. However, during drying process, the RL-95 was found to be more effective than the RG-50 and RM-75 in draining water.

Ogunro et al. (2005) investigated the effect of hydraulic flux on the stability of capillary barrier interface using soil columns. The soil columns consist of silt loam as the fine-grained layer and gravel as the coarse-grained layer. Water flow in the range of 1.57 – 55.5 cm/day was applied from the top of the soil columns. After 200 h, the tests was dismantled and analyzed for mobilized particles. The results showed that colloidal particles were transported from the fine-grained layer into the coarse-grained layer. The mobilized colloids do not have much impact on the stability of the interface. Under high flux rate, particles larger than colloids were also transported. The migration of these large particles was mainly concentrated in the upper portion of the gravel layer. Particle analysis of the silt loam (fine-grained layer) after the infiltration tests showed that there was no significant particle redistribution in the layer. Hence, particle mobilization can be modeled as a phenomenon that essentially occurred in the interface of fine-grained and coarse-grained layer.

2.4.3.3 Field tests

Other than laboratory experiments, several researchers also investigated the capillary barrier system in the field. Jelinek (1995) constructed a capillary barrier system on the landfill 'Am Stempel' at Marburg, Germany over an area of 3,300 m². The capillary barrier system consists of 40 cm of fine-grained layer overlying a 30 cm coarse-grained layer. A water balancing layer of 2 m thickness was constructed above the capillary barrier system to reduce the infiltration into the fine-grained layer by means of runoff, evapotranspiration and water storage. To measure the water balance of the capillary barrier system, a compartment of 10 m width was totally enclosed in PEHD-Liners at its side and bottom. The performance of the capillary barrier system was studied by investigating the water balance of the system. The maximum discharge from the fine-grained layer was 2.5 mm/d while

the discharge from the coarse-grained layer did not exceed a total of 3 mm in the whole year, indicating a good performance of the capillary barrier system.

Khire et al. (1999) conducted a field capillary barrier test at the Greater Wenatchee Regional Landfill in East Wenatchee, Washington. The test section was 30 x 30 m, of which 18.3 x 12.2 m region was used for monitoring. The capillary barrier consisted of a 15 cm uncompacted, sparsely vegetated sandy silt and a 75 cm clean, uniformly graded medium sand as a coarse grained layer (Figure 2.12). The surface layer was very thin because their objective was to determine if the capillary barrier effect and the water balance of the cover could be modeled. Thus, a thin surface layer was used to ensure that water would penetrate the coarse-grained layer and measurable percolation would occur. The results of the field capillary barrier tests were compared with the results of numerical simulation using the UNSAT-H model. It was concluded that the field capillary barrier performed in reasonable agreement with the theory under natural meteorological conditions.

Figure 2.12 Cross section of the instrumented test section (Khire et al., 1999)

A monitoring of an experimental soil cover was carried out by Yanful et al. (1999). The soil cover consisted of a compacted till barrier soil placed between gravel layers, and a final topsoil cap in one half and a coarse stone cap in the other half. The purpose of the upper gravel layer was to provide capillary break that would minimize percolation into the till, while the lower gravel layer was intended to reduce evaporation in the till during dry periods. The results showed that the compacted till desiccated more under the top soil than under the coarse stone because of the capillary barrier effect created by the fine top soil overlying the gravel layer. Because of this capillary barrier effect, water could not infiltrate into

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the compacted till, thus the till was not able to rebound to its pre-desiccation water content.

A large capillary barrier field test was conducted at the central landfill of Breinermoor, Germany. Von der Hude et al. (2000) constructed a capillary barrier as a landfill cover in area of approximately 100m x 220m. The materials used are locally available sand as the fine-grained layer and fine gravel as the coarse-grained layer. Several conclusions regarding the construction of capillary barrier in the field were pointed out, such as: conventional machine for earthworks can be used to construct capillary barrier, water collection system have to be made before the construction of capillary barrier layers, and protection against airborne sand must be highly considered.

2.4.4 Geocomposites capillary barrier system

The use of capillary barriers as soil cover has been studied widely. Field tests, laboratory tests, and numerical simulations have successfully demonstrated the effectiveness of this concept. The capillary barrier system will perform well if the appropriate combinations of materials are used. Stormont and Ramos (2000) pointed out that there are some limitations regarding the materials of capillary barrier, including: (1) the materials for unsaturated drainage system are often not readily available at the site and thus can be costly, and (2) the materials can be difficult to place on many slopes and locations. Therefore, an alternative material for capillary barrier system is needed. Stormont and Ramos (2000) selected geosynthetic materials to be the alternative material for capillary barrier system because it has several advantages as compared to soil-based system, such as:

1. Desirable properties can be optimized through appropriate design and controlled by manufacturer.
2. Drainage functions can be combined with other functions such as reinforcement and soil retention
3. A geosynthetic system will be thinner (approximately 2 cm thick vs. 30 cm thick for a soil-based system), minimizing its impact in the overall project design.

4. Geosynthetics can be readily delivered throughout the world.

Stormont (1998) investigated the application Geocomposite Capillary Barrier Drain (GCBD) system as a drainage system on a pavement design. The GCBD system was expected to be effective when placed between a base course and sub-grade by preventing the sub-grade from wetting process. The GCBD system consisted of three layers that are, from top to bottom: a transport layer, a capillary barrier layer, and a separator layer. Non-woven geotextiles can be used as a transport layer and a separator layer, while a geonet with relatively large, open pores functions as a capillary break. A laboratory setup to measure the GCBD system performance was constructed. A plexiglass box of 30 cm wide, 300 cm long, and 45 cm deep was filled with soil and the GCBD system (Figure 2.13). The box could be tilted at various angles and equipped with a system for adding water slowly to the top of the soil. Tensiometers were placed within the soil both above and below the drainage system to obtain insight into the performance of the GCBD system. Water which laterally drained from the system and water which moved downward through the GCBD system were collected. The tests results showed that commercially available geosynthetics, when combined in GCBD system, could drain water laterally while the adjacent soil remained unsaturated. As expected, the lateral drainage increased if the slope angle increased.

Figure 2.13 Schematic diagram of test device for measuring lateral drainage of geosynthetic unsaturated soil drainage system (Stormont, 1998)

Although this GCBD system seems similar to a conventional geocomposite drain, a GCBD was designed to drain water in the geotextile transport layer (not in the

geonet) under negative water pressures. A GCBD did not require an underlying impermeable layer as a conventional drain requires. In the GCBD system, the most important properties were the unsaturated hydraulic properties of the geosynthetic materials.

Another research by Henry and Holtz (2001) was conducted to investigate the effectiveness of geocomposite capillary barrier to reduce frost heave in soils. The experiments were conducted by freezing up-right cylindrical soil specimens with geosynthetics placed in them. The results showed that a geotextile failed as a capillary barrier and did not reduce the frost heave, while a geocomposite comprising needle-punched polypropylene geotextiles sandwiching a drainage net, was able to reduce frost heave when the suction head in the soil overlying the geocomposite was higher than 1.8 m.

In this study, the use of geocomposite in a capillary barrier system for slope stabilization against rainfall-induced slope failure will be investigated using a one-dimensional infiltration column and a two-dimensional sloping capillary barrier model.

2.5 Flux Boundary Conditions

A flux boundary is a surface boundary where flow or flux occurs. In the context of this study, the flux boundary is a ground surface, which water can flow across it. Water enters the soil by means of rainfall infiltration and water is removed from the soil by means of evapotranspiration (evaporation and transpiration). One of the objectives of this study is to investigate the effect of evaporation on the recovery of water storage in a capillary barrier system. Therefore, more emphasis is given on the review of the current literature with respect to evaporation from the ground surface.

2.5.1 Infiltration

The infiltration process in a layered soil has been investigated using several methods such as analytical solutions, numerical simulations, laboratory experiments, and field experiments. Srivastava and Yeh (1991) proposed analytical

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solutions for a one-dimensional, transient infiltration toward the water table in homogeneous and layered soils. The analytical solutions from Srivastava and Yeh (1991) were used by Romano et al. (1998) to validate their numerical model. Van Dam and Feddes (2000) conducted a numerical analysis using the backward finite difference method with special attention given to the selection of a head or flux controlled top boundary condition during the iterative solution of the Richards equation. The works by Srivastava and Yeh (1991), Warrick et al. (1991), Romano et al. (1998), Corradini et al. (2000), and Van Dam and Feddes (2000) show significant improvements in the methods of solving the Richards' equation as their models were mainly for the analysis of water flow in unsaturated soil.

Other than analytical solutions and numerical simulations, laboratory and field tests were also carried out to study the infiltration process in a soil. Two-dimensional infiltration tests to study the infiltration characteristics and wetting front development in a fine-grained soil over a coarse-grained soil were conducted by Miyazaki (1988). It was observed that the wetting fronts were parallel to the surface of the slope and the water flow was diverted laterally along the fine-coarse interface. Zhang et al. (2000) conducted a field infiltration tests on a residual soil and decomposed granite at a hillside in Hubei Province, China. The results showed that the infiltrating water was diverted laterally due to the presence of a joint. Thus, it was suggested that the discontinuities in a soil should be accounted for to describe a realistic infiltration process in a weathered soil.

In this study, the infiltration process is associated with the performance of a capillary barrier system during rainfall events. It is expected that water will be diverted laterally along the fine-coarse interface and breakthrough to the coarse-grained layer can be avoided.

2.5.2 Evaporation

This section explains the definition of evaporations, including potential and actual evaporations. Methods for evaluating evaporations are also summarized in this section.

2.5.2.1 Definition of potential and actual evaporations

The international Glossary of Hydrology (World Meteorological Organization 1974) defines potential evaporation as “The quantity of water vapour which could be emitted by a surface of pure water per unit surface area and unit time under existing atmospheric conditions.” In the simplest term, potential evaporation is defined as the upper limit or maximum rate of evaporation (Wilson et al., 1997), whereas actual evaporation can be defined as the removal of water from a soil at the ground surface. Unlike potential evaporation, which is independent of soil properties, actual evaporation begins to decline as a soil becomes unsaturated.

Figure 2.14 shows a typical relationship for the ratio of actual and potential evaporations (AE/PE) with water availability for sand and clay. Holmes (1961; after Wilson, 1997) showed that AE is equal to PE for both sand and clay soils when the water content is high or near field capacity. The AE/PE ratio for both soils start to decline as the availability of water decreases to the permanent wilting point for plants. The decline in this ratio varies significantly depending on the soil texture and drying rate.

Figure 2.14 Typical drying curves for sand and clay (Holmes, 1961; after Wilson, 1997)

2.5.2.2 Evaporation from a soil surface

Evaluation of evaporation from a soil surface is approximately the same with evaporation from water surface if the supply of water to the ground surface is unlimited. However, if the soil starts to dry out, the evaluation of evaporation from the soil surface is getting complex. Hillel (1980) pointed out that there are three conditions that must exist for the process of evaporation from a soil surface, which are:

1. A continuous supply of energy must be available for the latent heat of vaporization.
2. The vapour pressure of the atmosphere above the soil surface must be less than the vapour pressure at the soil surface.
3. A continuous supply of water to the evaporated area is required.

According to Hillel (1980), evaporation from a soil surface can be divided into three distinct phases, which are:

1. The initial constant-rate stage
2. An intermediate falling-rate stage
3. A residual slow-rate stage

Figure 2.15 shows the three stages of evaporation from a soil surface. The constant-rate stage occurs when the soil is at near saturation or wet conditions. At the wet conditions, the soil is sufficiently conductive to supply water to the evaporative surface. Thus, the actual evaporation rate is the same with the potential evaporation, which is controlled by the climatic conditions. If there is no recharge from the groundwater, the surface water content decreases gradually. Eventually, at one point, the soil is no longer conductive to supply water to the evaporative surface and the evaporation rate started to decrease. The actual time and water content at which the evaporation rate started to decrease depends on the initial evaporation rate and the soil properties. The falling-rate stage occurs when the soil can not transmit water into the evaporating surface due to its low coefficient of permeability. The coefficient of permeability of the soil decreases as the soil becomes unsaturated. The slow-rate stage of evaporation occurs when the surface

zone becomes sufficiently desiccated such that the liquid phase of the soil water is discontinuous (Hillel, 1982, after Wilson, 1990). Water molecules migrate to the evaporative surface only through water vapour diffusion and the evaporation rate is controlled by the vapour diffusivity of the dried surface.

Figure 2.15 Three stages of evaporation from soil (Hillel, 1980; after Wilson, 1990)

2.5.2.3 Traditional methods for evaluating evaporation

Wilson (1990) classified methods for evaluating evaporation into three groups, which are:

1. Direct measurement methods
2. Micrometeorological methods
3. Regional climatological methods

Some methods from the above groups which are relevant to this study will be discussed below.

Evaporation Pan

Evaporation pan is one example of the device for direct measurement of potential evaporation. Most likely, evaporation pan is the most widely used methods for direct measurement of potential evaporation. Other than for measuring the potential evaporation, the evaporation pan can also be used to estimate the actual evaporation. Evaporation pan gives realistic estimate of potential

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evapotranspiration in humid regions (Rosenberg et al., 1983; after Wilson, 1990). The rate of the actual evapotranspiration is typically 60 to 90 percent of the rate from the evaporation pan measurement.

Mass Transfer Method

The mass transfer method is one of the most used micrometeorological methods. The mass transfer approach has the deepest historical roots as its origin is frequently attributed to Dalton at the turn of the nineteenth century (Gray, 1970; Rosenberg et al., 1983; after Wilson, 1990). In 1802, Dalton proposed a mass transfer equation to evaluate evaporation from a water surface (i.e., potential evaporation), which can be written as follows:

$$PE = f_v (\bar{u}_{v0}^w - \bar{u}_v^a) \quad (2.1)$$

where:

PE = potential evaporative flux

f_v = transmission function that depends on the mixing characteristics of the air in the atmosphere above the evaporating surface

$\bar{u}_{v0}^w$ = saturation vapour pressure of water at the temperature of the water surface

$\bar{u}_v^a$ = vapour pressure of the air in the atmosphere above the water surface

Generally, the f_v function is evaluated based on an empirical method. Nevertheless, some researchers also evaluated the f_v function using an aerodynamic approach.

The mass transfer method provides good estimates of the evaporative flux when the relative humidity of the evaporating surface is 100% and the surface temperature is known. Difficulties arise when the availability of water at the surface is decreased. The actual vapour pressure at the surface drops to a value below the saturated vapour pressure (Rosenberg et al., 1983; after Wilson, 1990).

The Penman Method

One example of the regional climatological method is the Penman method. According to Rosenberg et al. (1983) Penman method is the most popular and

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widely used method for evaluating the potential evapotranspiration. This is due to its simplicity and the ease of application. The Penman method only requires the measurement of weather parameters such as air temperature, relative humidity, and wind speed.

The Penman method is based on the energy balance and the mass transfer equations. The formula proposed by Penman can be written as follows:

$$E = \frac{\Gamma Q + v E_a}{\Gamma + v} \quad (2.2)$$

where:

E = potential evaporation per unit time

E_a = $f_v (\bar{u}_{v0}^w - \bar{u}_v^a)$

$\bar{u}_{v0}^w$ = saturation vapour pressure of water at the temperature of the water surface

$\bar{u}_v^a$ = vapour pressure of the air in the atmosphere above the water surface

f_v = transmission function that depends on the mixing characteristics of the air in the atmosphere above the evaporating surface

= $0.35 (1 + 0.146 W_a)$

W_a = wind speed (km/h)

Q_n = all net radiation at the soil surface

Γ = slope of the saturation vapour pressure versus temperature curve at the mean temperature of the air

v = psychrometric constant

The Penman formula is most suitable for evaluating the evaporation from an open water surface. However, Penman extended the formula to include bare soil and grass surfaces with a plentiful water supply. The evaporation from a wet bare soil is 0.9 times that from an open water surface exposed to the same weather conditions in all seasons (Penman, 1948; after Wilson, 1990).

2.5.2.4 Recent development on the methods of evaluating evaporation from a soil surface

The actual evaporation from a soil surface is greatly reduced relative to the potential evaporation rate. Some researchers had proposed several different approaches to evaluate the actual evaporation from a soil surface. Some approaches that are relevant to this study will be presented in this section.

Wilson (1990) proposed a theoretical model for predicting the rate of evaporation from a soil surface. The model was based on a system of equations for coupled heat and mass transfer in soil. The flow of liquid water and water vapour were described using the Darcy's Law and Fick's Law, respectively. Dalton's mass transfer law was used to calculate the rate of evaporation based on the suction at the soil surface. The theoretical model was then used to predict soil evaporation rates, water content profiles, and temperature profiles for a controlled column evaporation test. The values computed by the model agreed well with the values measured from the column evaporation test.

In addition to the theoretical model, Wilson (1990) also modified the Penman method to calculate the actual evaporation from unsaturated soil surfaces. The modified Penman method can be written as follows:

$$AE = \frac{\Gamma Q + vE_a}{\Gamma + vA} \quad (2.3)$$

where:

AE = actual evaporation per unit time

E_a = $f_v \cdot \bar{u}_v^a (B - A)$

B = inverse of the relative humidity in the air

A = inverse of the relative humidity at the soil surface

The fundamental assumption made in the formulation of Eq. 2.3 is that the saturation vapour pressure of water at the temperature of the water surface ($\bar{u}_{v0}^w$) in Eq. 2.1 can be replaced by the actual vapour pressure at the soil surface.

Stewart and Broadbridge (1998) calculated the relative humidity at the soil surface as a function of time using a numerical method which has been validated against the analytically solvable model. The results showed that the surface relative humidity suddenly drops at a particular time, while the surface water content decreases gradually. Nevertheless, Stewart and Broadbridge (1998) concluded that the gradual decrease in surface water content is not inconsistent with a sharp, step-like decrease in relative humidity.

Yanful and Mousavi (2003) proposed an analytical method to estimate the falling-rate evaporation based on the equation proposed by Gardner and Hillel (1962). The required data for the analysis are the soil-water characteristic curve, permeability function, and initial water content. The applicability of the equation was assessed by comparing results from the equation to the results obtained experimentally and numerically using the finite element software SoilCover. It was concluded that the falling-rate evaporation from a finite soil column can be estimated using Gardner and Hillel (1962) equation.

Recent works by Lee et al. (2003) proposed a theoretical model to predict the evaporation rate from surface of a deformable soil. The model was based on equations for coupled heat and mass transfer in unsaturated soils derived by Wilson et al. (1993). The results showed that the proposed theoretical model, which considers deformation during drying, could simulate the evaporation rate as verified by a column drying test.

2.6 Shrinkage Characteristics

Several research works in geotechnical engineering discipline on shrinkage and cracking behaviour are presented in this section. In 1976, Fredlund and Morgenstern proposed a consistent framework for unsaturated soils, which represents total stress as a function of pore-air pressure ($\sigma - u_a$) and matric suction ($u_a - u_w$). As soil is desaturated when it cracks, Lau (1987) made use of the unsaturated soil stress state variables and linked the variables using linear elastic theory under zero lateral strain condition to predict depth of cracks.

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Kodikara et al. (2002) presented a synthesis of mechanisms related to soil behaviour during drying and wetting cycles and shrinkage cracking. It was pointed out that there are two types of boundary conditions for the shrinkage cracking, which are unrestrained shrinkage and restrained shrinkage. Unrestrained shrinkage means that the soil can shrink without being subjected to any restraint. This condition can be achieved in the laboratory under controlled condition. Under this condition, shrinkage cracking cannot develop because tensile stresses and strains cannot develop in soil under unrestrained condition. The second condition, restrain shrinkage, is generally applicable to practical conditions where either internal or external restraint is applied to the soil.

A new method to estimate the vertical movement of a ground surface for soil that swells and shrinks due to variations in water content was proposed by Briaud et al. (2003). To obtain the parameters needed for the analysis, a shrinkage test was carried out. The shrinkage test was carried out using a cylindrical soil specimen of 75 mm diameter and 150 mm high. The specimen was dried at room temperature to avoid or limit the amount of soil cracking that could occur if the drying rate was too fast. The size and weight of the specimen were measured periodically during the drying process. It was recommended to measure the specimen every hour for the first 8 hours of the drying process.

Similar shrinkage tests were conducted in this study. The shrinkage tests were carried out to obtain the shrinkage characteristics of the residual soil and the mixtures of the residual soil and fine sand. It is desirable to obtain a soil with low shrinkage behaviour and good drainage capability to be used as a fine-grained layer in a capillary barrier system.

A study on the shrinkage curve was conducted by Fredlund et al. (2002). A mathematical relationship to model the shrinkage behaviour of slurried, compacted, and undisturbed soils were developed. The proposed equation to represent the shrinkage curve is given as follows:

$$e(w) = a_{sh} \left[\frac{w^{c_{sh}}}{(b_{sh})^{c_{sh}}} + 1 \right]^{\frac{1}{c_{sh}}} \quad (2.4)$$

where:

$$\begin{aligned} a_{sh} &= \text{minimum void ratio, } e_{min} \\ b_{sh} &= \text{slope of the line of tangency} \\ c_{sh} &= \text{curvature of the shrinkage curve} \\ a_{sh}/b_{sh} &= G_s/S = \text{constant for a specific soil} \end{aligned}$$

The above equation will be used to fit the experimental data obtained in this study.

Fredlund et al. (2002) also suggested that the volume change function between void ratio and soil suction could be obtained by combining the shrinkage curve and soil-water characteristic curve.

2.7 Concluding Remarks

Capillary barrier system as a soil cover for landfill in arid and semi-arid region has been investigated. Recent research work has also studied the mechanism of sloping capillary barrier system under high rainfall intensities. The research work shows that capillary barrier has potential to be used as a soil cover for slope stabilization against the rainfall-induced slope failure. In order to implement capillary barrier system for slope stabilization in the field, availability of materials for capillary barrier system has to be considered. The potential use of local residual soil as a fine-grained layer in capillary barrier system has not been fully studied. The effects of evaporation on the recovery of water storage in capillary barrier using residual soil (clayey or silty soil) as the fine-grained layer need to be investigated in order to select the appropriate material for the capillary barrier system.

Geosynthetic material has also been considered as a capillary barrier material due to its characteristics and worldwide availability. The application of geosynthetic capillary barrier system in the pavement design has been studied. However, its application for slope stabilization against rainfall-induced slope failure has never been fully investigated. Therefore, this study was conducted to investigate the performance of capillary barriers constructed using local residual soil and geosynthetic material.

Chapter 3

THEORETICAL CONSIDERATIONS

3.1 Introduction

This chapter presents the theories that are applicable to this research. As the mechanism of a capillary barrier system is based on the unsaturated soil mechanics, the theories that are presented in this chapter are generally associated with unsaturated soil mechanics. The introduction to unsaturated soil mechanics is presented in Section 3.2. Subsequently, theories related to soil suction is presented in Section 3.3. Section 3.4 explains the theories and equations on the soil-water characteristic curve. Water flow in unsaturated soils under an isothermal condition and non-isothermal conditions are presented in Sections 3.5 and 3.6, respectively. Finally, methods to obtain permeability function of unsaturated soil are explained in Section 3.7.

3.2 Introduction to Unsaturated Soil Mechanics

According to Fredlund and Rahardjo (1993a), an unsaturated soil can be defined as a soil which has four phases (i.e., solid, water, air, and contractile skin) and the pore-water pressure is negative relative to the pore-air pressure. The pore-water pressure in unsaturated soil is highly influenced by the flux boundary condition changes (i.e., infiltration and evaporation) caused by the changes in climatic conditions.

Figure 3.1 shows the classification of the regions within a saturated-unsaturated soil based on its degree of saturation. The portion of the soil above the groundwater table is called as the vadose zone and considered as unsaturated. As shown in the figure, unsaturated soils have a wide range of degree of saturation. It can be near to 100% saturation in the capillary fringe zone and completely dry at the ground surface. According to Fredlund (2000), the behavioural science for an unsaturated soil has been primarily developed for the case where the water and air

phases are continuous (i.e., two-phase zone) and the degree of saturation is generally in the range of 20% to 80%.

Figure 3.1 Classification of the regions within a saturated-unsaturated profile (modified from Fredlund, 2000)

Three possible negative pore-water pressure profiles in the vadose zone are illustrated in Figure 3.2. Line 1 defines the pore-water pressure profile under static equilibrium conditions with respect to the water table. Its magnitude is equal to the gravitational head measured relative to the water table. In other words, the hydraulic head (i.e., the gravitational head plus the pore-water pressure head) is equal to zero throughout the soil profile. Hence, the hydraulic gradient is zero, meaning that there can be no water flow in the vertical direction. Line 2 indicates the conditions where the pore-water pressure becomes more negative due to the steady-state evaporation. The hydraulic head has a nonlinear distribution from a zero value at the water table to a more negative value at the ground surface. As water flows from a high hydraulic head to a lower hydraulic head, in this case, water flows upward from the water table to the ground surface. The upward constant flux of water is designated as positive for a steady-state evaporation.

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The condition of a steady-state infiltration is illustrated by line 3. The negative pore-water pressure increases from the static equilibrium condition. The hydraulic head at the ground surface is positive and decreases to zero at the water table. Therefore, water flows downward with a constant, negative flux for a steady state infiltration.

Figure 3.2 Static equilibrium and steady-state flow conditions in the zone of negative pore-water pressures (Fredlund and Rahardjo, 1993a)

3.3 Theory of Soil Suction

Soil suction is commonly referred to as the free energy state of soil water (Edelfsen and Andersen, 1943; after Fredlund and Rahardjo, 1993a). Richards (1965) pointed out that the free energy of the soil water can be measured in terms of the partial vapour pressure of the soil water. The relationship between soil suction and the partial pressure of the pore-water vapour is presented in the following equation:

$$\psi = -\frac{RT}{v_{w0} \cdot \omega_v} \ln \left(\frac{\bar{u}_v}{\bar{u}_{v0}} \right) \quad (3.1)$$

where:

ψ = soil suction or total suction (kPa)

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- R = universal (molar) gas constant [i.e., 8.31432 J/(mol K)]
 T = absolute temperature [i.e., $T = (273.16 + t^\circ)$ (K)]
 t° = temperature ($^\circ\text{C}$)
 v_{w0} = specific volume of water or the inverse of the density of water [i.e., $1/\rho_w$ (m^3/kg)]
 ρ_w = density of water (i.e., 998 kg/m^3 at $t = 20^\circ\text{C}$)
 ω_v = molecular mass of water vapour (i.e., 18.016 kg/kmol)
 $\bar{u}_v$ = partial pressure of pore-water vapour (kPa)
 $\bar{u}_{v0}$ = saturation pressure of water vapour over a flat surface of pure water at the same temperature (kPa)

Equation 3.1 shows that the reference state for quantifying the components of suction is the vapour pressure above a flat surface of pure water. The term $\bar{u}_v / \bar{u}_{v0}$ is referred to as relative humidity, h_s (%). Figure 3.3 shows a plot of Eq. 3.1 for three different temperatures. It can be seen that the soil suction, ψ , is equal to zero when the relative humidity, h_s , is equal to 100%. A relative humidity value less than 100% in a soil indicates the presence of suction in the soil. The range of suctions of interest in geotechnical engineering commonly corresponds to high relative humidities.

Figure 3.3 Relative humidity versus total suction relationship (Fredlund and Rahardjo, 1993a)

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The soil suction is commonly known as total suction and has two components, namely, matric suction and osmotic suction. The relationship between total, matric, and osmotic suctions can be written in the following equation:

$$\psi = (u_a - u_w) + \pi \quad (3.2)$$

where:

- ψ = soil suction or total suction (kPa)
- $(u_a - u_w)$ = matric suction (kPa)
- u_a = pore-air pressure (kPa)
- u_w = pore-water pressure (kPa)
- π = osmotic suction (kPa)

The matric suction is defined as the difference between pore-air pressure, u_a , and pore water pressure, u_w . It is commonly associated with the capillary phenomenon arising from the surface tension and can be described using the capillary model as shown in Figure 3.4.

Figure 3.4 Physical model and phenomenon related to capillarity (Fredlund and Rahardjo, 1993a)

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Points A, B, and C in the capillary system shown in Figure 3.4 are in a hydrostatic equilibrium. The elevation of points A and B (water surface) is considered as the datum for the system. The water pressure at points A and B are atmospheric (equal to zero). Thus, the hydraulic heads at points A and B are equal to zero. Point C is located at a height of h_c from the datum (i.e., elevation head is equal to h_c). Since points A, B, and C are in a hydrostatic equilibrium, the hydraulic heads at all three points should be equal (in this case is zero). This means that the pressure head at point C is equal to the negative value of its elevation head. The water pressure at point C can be calculated as $u_w = -\rho_w g h_c$. As the air pressure at point C is atmospheric ($u_a = 0$), thus the matric suction, $(u_a - u_w)$, at point C can be expressed as: $(u_a - u_w) = \rho_w g h_c$.

Other than matric suction, the component of total suction is osmotic suction. It is a component of total suction which is related to the salt content in the pore-water. If the salt content in a soil changes, the overall volume and shear strength of the soil will also change. In other words, osmotic suction changes have an effect on the mechanical behavior of a soil. Osmotic suction is present in both saturated and unsaturated soils. Therefore, the role of osmotic suction is equally applicable to both unsaturated and saturated soils.

Most engineering problems in unsaturated soils are commonly results of environmental changes. These changes primarily affect the matric suction. Figure 3.5 shows that the changes in osmotic suction are less significant than the changes in matric suction.

3.4 Soil-water Characteristic Curve

A soil-water characteristic curve (SWCC) for a soil is defined as the relationship between water content and suction of the soil (Williams, 1982 after Fredlund and Xing, 1994). The water content defines the amount of water in the pores of the soil. In geotechnical engineering, the amount of water in the soil generally quantified in terms of gravimetric water content, w , while in soil science, volumetric water content, θ_w , is most commonly used. The gravimetric water content, w , is defined as the ratio of the mass of water, M_w , to the mass of soil solids, M_s , while the

volumetric water content, θ_w , is defined as the ratio of the volume of water, V_w , to the total volume of the soil, V . The suction can be either matric suction, $(u_a - u_w)$, or total suction, ψ . Fredlund and Xing, 1994 suggested that SWCC be used to represent the relationship between volumetric water content, θ_w , and matric suction, $(u_a - u_w)$.

Figure 3.5 Total, matric, and osmotic suction measurements on compacted Regina clay (Krahn and Fredund, 1972; after Fredlund and Rahardjo, 1993a)

Figure 3.6 shows the drying and wetting SWCC with their key characteristics, such as: saturated volumetric water content, θ_s , residual volumetric water content, θ_r , air-entry value, ψ_a , and water-entry value, ψ_w . The saturated volumetric water content, θ_s , is the water content at saturated condition (at very low matric suction); while the residual water content, θ_r , is the water content at which a large suction change is required to remove additional water from the soil. The air-entry value, ψ_a , of a soil is the matric suction value where air starts to enter the largest pore in the soil; while the water-entry value, ψ_w is the matric suction value where water starts to enter the soil during wetting process. Typical soil-water characteristic curves for different types of soil are shown in Figure 3.7.

Figure 3.6 Definition of terms for a typical soil-water characteristic curve (Fredlund and Xing, 1994)

Figure 3.7 Typical soil-water characteristic curves for a sandy soil, a silty soil, and a clayey soil (Fredlund and Xing, 1994)

Various equations have been proposed to represent SWCC. The commonly used equations are the Brooks and Corey (1964), Van Genuchten (1980), and Fredlund and Xing (1994). Leong and Rahardjo (1997a) evaluated numerous SWCC equations and showed that all the equations can be derived from a single generic form. Leong and Rahardjo (1997a) concluded that Fredlund and Xing (1994)

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equation gave the best fit among other equations and recommended that Fredlund and Xing equation with the correction factor $C(\psi) = 1$ be used for the fitting of SWCC. The equation proposed by Fredlund and Xing (1994) can be written as follows:

$$\theta_w = C(\psi) \frac{\theta_s}{\left[\ln \left(e + \left(\frac{\psi}{a} \right)^n \right) \right]^m} \quad (3.3)$$

where:

- $C(\psi)$ = correction factor
- = 1 (as suggested by Leong and Rahardjo, 1997a)
- θ_w = volumetric water content
- θ_s = saturated volumetric water content
- ψ = Matric suction (kPa)
- a = fitting parameter related to the air-entry value of the soil (kPa)
- n = fitting parameter related to the slope of the SWCC
- m = fitting parameter related to the residual water content
- e = natural number, 2.71828...

The soil-water characteristic curve data obtained in this study were fitted using The Fredlund and Xing (1994) equation with the correction factor $C(\psi) = 1$ as suggested by Leong and Rahardjo (1997a)

Hysteresis behaviour of Soil-water Characteristic Curve

Hysteresis behavior of soil-water characteristic curve (SWCC) is caused by non-uniformity of pore size distribution and the presence of air in the soil. The hysteresis behavior can be explained using the ink bottle effect as shown in Figure 3.8.

The wetting path of a SWCC can be described by placing an empty capillary tube in a water bath. Water will rise up inside the tube until the equilibrium condition is reached. On the other hand, the drying path is occurred when a capillary tube

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which is already filled up with water is placed in the water bath. Water will be drained out until it reaches the equilibrium condition. When the tube has a uniform radius, the equilibrium conditions achieved from the wetting and drying paths are similar (Figure 3.8 a). However, if the tube has a bulb with a radius larger than the tube's radius, the equilibrium conditions achieved from the wetting and drying paths will be different (Figure 3.8 b and 3.8 c). Figure 3.8 b shows that the presence of the bulb prevents water from rising up beyond the base of the bulb in a wetting path. However, in the drying path, the bulb is still filled with water since the bulb has already been filled with water initially (Figure 3.8 c).

Figure 3.8 Ink bottle effect in capillary tube (Taylor, 1984; after Fredlund and Rahardjo, 1993a)

Figure 3.9 shows the hysteresis behavior of soil-water characteristic curves. It can be seen that for a given matric suction, the volumetric water content on the wetting curve is always lower than that on the drying curve. The drying curve that starts from the saturation condition is called as the primary drying curve. In the same manner, the wetting curve that starts from a dry condition is called as the primary wetting curve. Other than the primary curves, there are curves that start from a particular condition which is neither saturated nor dry condition. These curves are called the scanning curves. In this study, the scanning curves will be used as an indicator to evaluate the recovery of a capillary barrier system during the drying process.

Figure 3.9 Primary and scanning curves of the soil-water characteristic curve

3.5 Water Flow in Unsaturated Soils under an Isothermal Condition

Similar to a saturated soil, the flow of water in an unsaturated soil is also governed by the hydraulic head gradient. The hydraulic head at a point in the water phase is equal to the total energy at the point divided by its unit weight. There are three components of energy at a point in the water phase, namely, gravitational, pressure, and velocity components as given in the following equation:

$$E = M_w g y + \frac{M_w u_w}{\rho_w} + \frac{M_w v_w^2}{2} \quad (3.4)$$

The hydraulic head at a point is equal to the energy per unit weight. It is obtained by dividing Eq. 3.4 by the weight of water (i.e., $M_w g$):

$$h_w = y + \frac{u_w}{\rho_w g} + \frac{v_w^2}{2g} \quad (3.5)$$

where:

h_w = hydraulic head or total head (m)

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- y = elevation from a datum (m)
 u_w = pore-water pressure (kPa)
 ρ_w = density of water (Mg/m³)
 g = gravitational acceleration (m/s²)
 v_w = velocity or flow rate of water (m/s)

As given in Eq. 3.5, the hydraulic head consists of three components, namely, the gravitational head, y , the pressure head ($u_w/\rho_w g$), and the velocity head ($v_w^2/2g$). The velocity head of the water flow through a soil is negligible as compared to the gravitational and the pressure heads. Therefore, Eq. 3.5 can then be simplified to become:

$$h_w = y + \frac{u_w}{\rho_w g} \quad (3.6)$$

The water flows in saturated and unsaturated soils are governed by the gradient of the hydraulic head. The hydraulic head gradient, i , is defined as the ratio of the difference in the hydraulic head, Δh_w , to the length of the flow path, Δl :

$$i = \frac{\Delta h_w}{\Delta l} \quad (3.7)$$

Darcy's law

The flow of water in a saturated soil is described using Darcy's law. In 1856, Darcy formulated that the rate of water flow through a soil mass is proportional to the hydraulic head gradient:

$$v_w = -k_w \frac{\partial h_w}{\partial y} = -k_w i \quad (3.8)$$

where:

- v_w = discharge velocity (m/s)
 k_w = coefficient of permeability (m/s)
 $\partial h_w / \partial z = i$ = hydraulic head gradient in y -direction

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The negative sign in Eq. 3.8 indicates that water flows in the direction of a decreasing hydraulic head.

Later research works by Buckingham (1907), Richards (1931), and Childs and Collis-George (1950) showed that Darcy's law can also be applied to the flow of water through unsaturated soils. However, in unsaturated soils, the coefficient of permeability cannot be assumed to be constant. The coefficient of permeability varies with the changes in water content or matric suction. Generally, the coefficient of permeability decreases as the matric suction increases. The relationship between coefficient of permeability and matric suction is known as the permeability function (Section 3.2.4).

Governing equation for water flow through unsaturated soil

Fredlund and Rahardjo (1993a) presented the governing equation for an unsteady state water seepage through an isotropic soil as follows:

$$\frac{\partial}{\partial x} \left(k_w \frac{\partial h_w}{\partial x} \right) + \frac{\partial}{\partial y} \left(k_w \frac{\partial h_w}{\partial y} \right) = m_2^w \rho_w g \frac{\partial h_w}{\partial t} \quad (3.9)$$

where:

- k_w = coefficient of permeability with respect to the water phase for the x- and y-directions (i.e., isotropic soil)
- h_w = hydraulic head
- m_2^w = coefficient of water volume change with respect to a change in matric suction, $(u_a - u_w)$
- ρ_w = density of water
- g = gravitational acceleration

The left side of the equation represents the flow of water through a soil element in the x- and y-directions based on Darcy's law, while the right side of the equation represents the change in the volume of water in the soil element based on the constitutive equation of water phase.

3.6 Water and Water Vapour Flow in Unsaturated Soils under a Non-isothermal Condition

The equation for water and water vapour flows under a non-isothermal condition is similar to the equation for water flow in a one-dimensional consolidation with an additional term. The additional term was added to incorporate the water vapour flow due to diffusion and advection processes (Wilson, 1990). The one-dimensional fluid and vapour flow equation for the water phase in the y-direction is expressed as follows:

$$\frac{\partial u_w}{\partial t} = -C_w \frac{\partial u_a}{\partial t} + \frac{1}{\rho_w g m_2^w} \frac{\partial}{\partial y} \left(k_w \frac{\partial u_w}{\partial y} \right) + \frac{\bar{u}_a + \bar{u}_v}{\bar{u}_a} \frac{1}{\rho_w m_2^w} \frac{\partial}{\partial y} \left(D_v \frac{\partial \bar{u}_v}{\partial y} \right) \quad (3.10)$$

where:

C_w = interaction coefficient associated with the water phase partial differential equation, i.e., $(1 - m_2^w / m_{1k}^w) / (m_2^w / m_{1k}^w)$

u_a = pore-air pressure

u_v = partial pressure of water vapour in air

D_v = diffusion coefficient of the water vapour through the soil (kg·m/kN·s)

In this study, Eq. 3.9 and Eq. 3.10 are solved using the saturated/unsaturated finite element seepage modelling software, SVFlux (Soilvision System Ltd., 2005).

3.7 Permeability Function

Fredlund and Rahardjo (1993a) defined the coefficient of permeability with respect to the water phase as a measure of the space available for water to flow through soil. Unlike the coefficient of permeability of a saturated soil, which is constant, the coefficient of permeability of an unsaturated soil is not constant. It depends on the volumetric water content, θ_w , of the soil. As the volumetric water content, θ_w , in an unsaturated soil depends on the soil suction, it was suggested that the term “permeability function for unsaturated soils” be used to represent the relationship between the coefficient permeability and soil suction (Fredlund et al, 1994).

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There are two approaches to obtain the permeability function for unsaturated soils. The first approach is using empirical equations and the second is using statistical models. The empirical equations describe the permeability functions for an unsaturated soil as a function of suction, ψ , or as a function of volumetric water content, θ_w . It can be used to determine the permeability function when measured data for the relationship between the coefficient of permeability and suction, $k(\psi)$, or for the relationship between the coefficient of permeability and the volumetric water content, $k(\theta_w)$, are available. The minimum number of data required to determine the permeability function is equal to the number of fitting parameters adopted in the equations.

The statistical models determine the permeability function using the soil-water characteristic curve. This approach is based on the fact that the permeability function and the soil-water characteristic curve are primarily determined by the pore-size distribution of the soil. Figure 3.10 illustrates the relationship between the soil-water characteristic curves and the permeability functions for a sand and a clayey silt.

It can be seen that the coefficients of permeability, k_w , for both soils remain constant at the saturated coefficient of permeability, k_s , until the air-entry value, ψ_a , of the soil is reached. The coefficients of permeability for both soils decrease drastically as the matric suction value exceeds their air-entry value. The permeability functions are essentially linear until the residual matric suction is reached. Beyond this point, the coefficient of permeability appears to be constant but there is a lack of data to confirm this portion of permeability function (Fredlund, 2000).

In this study, the permeability function was calculated using the statistical method presented in Fredlund and Rahardjo (1993a), which can be summarized as follows. The SWCC is first divided into “m” equal intervals of volumetric water content. The first volumetric water content corresponds to saturated condition. Each volumetric water content midpoint $(\theta_w)_i$, corresponds to a particular matric suction, $(u_a - u_w)_i$. The following equation is used to calculate the $k_w(\theta_w)_i$:

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Figure 3.10 Relationship between soil-water characteristic curves and permeability functions for a sand and a clayey silt (Fredlund and Rahardjo, 1993b)

$$k_w(\theta_w)_i = \frac{k_s}{k_{sc}} A_d \sum_{j=i}^m \left\{ (2j+1-2i)(u_a - u_w)_j^{-2} \right\}$$

$$i = 1, 2, \dots, m \quad (3.11)$$

where:

- $k_w(\theta_w)_i$ = predicted water coefficient of permeability for a volumetric water content, $(\theta_w)_i$, corresponding to the i^{th} interval (m/s)
- i = interval number which increases as the volumetric water content decreases
- j = a counter from "i" to "m"

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- m = total number of intervals between the saturated volumetric water content and the lowest volumetric water content on the experimental SWCC
 k_s = Measured saturated coefficient of permeability
 k_{sc} = Saturated coefficient of permeability
 A_d = Adjusting constant
 $(u_a - u_w)_j$ = Matric suction corresponding to the j^{th} interval (kPa)

The $\sum_{j=i}^m \left\{ (2j+1-2i)(u_a - u_w)_j^{-2} \right\}$ term in Eq. 3.11 describes the shape of the permeability function, while the A_d term is used to scale the coefficient of permeability. The A_d term is a constant when predicting the coefficient of permeability and can be assumed to be unity. The unsaturated coefficient of permeability, k_w , are adjusted in accordance with the saturated coefficient of permeability, k_s , by use of the term (k_s/k_{sc}) . The value of k_{sc} is calculated by substituting matric suctions corresponding to the mid-points along the drying curve and A_d equal to 1 into the following equation.

$$k_{sc} = A_d \sum_{j=i}^m \left\{ (2j+1-2i)(u_a - u_w)_j^{-2} \right\}$$

$$i = 0, 1, 2, \dots, m \quad (3.12)$$

Based on the k_{sc} value calculated using the above equation, A_d equal to 1 and the laboratory measured k_s , the $k_w(\theta_w)_i$ was then calculated using Eq. 3.11.

Fredlund and Rahardjo (1993a) presented a comparison between the computed permeability functions and the permeability values measured by Gonzales and Adams (1980) as can be seen in Figure 3.11.

Instantaneous Profile Method

Other than the determination of permeability function based on the statistical method, another method to determine permeability function of the soils used in the capillary barrier models was adopted. Instantaneous profile method to determine the permeability function (Watson, 1966; Hillel et al., 1972 and Olson

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and Daniel, 1981; after Fredlund and Rahardjo, 1993a) was adopted due to the availability of pore-water pressure data along the capillary barrier models.

Figure 3.11 Comparison between the computed and measured coefficient of permeability (Fredlund and Rahardjo, 1993a)

The calculation procedure for the instantaneous profile used in this study is summarized as follows. The hydraulic heads, h_w (summation of the pore-water pressure head, h_p and gravitational head, h_g) at a one-dimensional cross-section in the capillary barrier model were plotted at specified elapsed times. The hydraulic head gradient, i_w , at a specific depth for a particular elapsed time can be calculated from the slope of the hydraulic head profile at that depth.

$$i_w = \frac{dh_w}{dz} \quad (3.13)$$

Figure 3.12 illustrates the hydraulic head profile in the capillary barrier model based on the measurements from tensiometers a and b during a drying process.

The total volume of water in the soil between two tensiometers (tensiometer a and tensiometer b) at a specific time is obtained by integrating the volumetric water content profiles:

Figure 3.12 Hydraulic head profiles measured by tensiometers a and b during a drying process

$$V_w = \int_a^b \theta_w(z) A dz \quad (3.14)$$

where:

- V_w = total volume of water in the soil between two tensiometers (tensiometer a and tensiometer b)
- $\theta_w(z)$ = volumetric water content profile as a function of depth, z , for a specific time
- A = cross-sectional area of the measured zone

The volumetric water content data used in the calculation was obtained from the measured pore-water pressure and the soil-water characteristic curves. The volumetric water content profiles corresponding to the pore-water pressure data measured by tensiometers a and b are presented in Figure 3.13.

The water flow rate at a point was calculated from the quantity of water flowing past the point between two consecutive times (dt). The water flow rate, v_w , is formulated as follows:

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Figure 3.13 Volumetric water content profiles corresponding to the pore-water pressure data measured by tensiometers a and b

$$v_w = \frac{dV_w}{Adt} \quad (3.15)$$

where:

- dV_w = difference in volumes of water computed between two consecutive times (dt)
 dt = time interval of the measurements

The coefficient of permeability, k_w , is computed by dividing the flow rate, v_w , by the average hydraulic head gradient, i_{ave} :

$$k_w = \frac{v_w}{i_{ave}} \quad (3.16)$$

where:

- i_{ave} = average value of hydraulic head gradients (i_w) at a point between two consecutive times used in the flow rate calculation

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The computations of the coefficient of permeability, k_w , were repeated for different times and different tests so that many coefficients of permeability corresponding to various water contents or suction values were obtained and produced the permeability function of the soil.

3.8 Concluding Remarks

The theories applicable to this study are presented in this chapter. Unsaturated soil properties which are necessary to study capillary barrier system, including soil-water characteristic curve and permeability function, are explained. The key parameters of drying and wetting SWCCs are saturated volumetric water content, θ_s , residual volumetric water content, θ_r , air-entry value, ψ_a , and water-entry value, ψ_w . Methods for the determination of permeability function, including the statistical method and the instantaneous profile method are described in this chapter. The instantaneous profile method is used to calculate the permeability function of the residual soil used in the capillary barrier model. This method is selected due to the availability of pore-water pressure data along the model. The governing equations for water flow in unsaturated soils under isothermal and non-isothermal conditions are presented for their use in numerical analyses.

Chapter 4

RESEARCH PROGRAMME

4.1 Introduction

In this chapter, the research programme is discussed in detail. The outline of research programme is explained in Section 4.2. In Section 4.3, investigations of material properties are described. The evaporation and shrinkage tests set-up are presented in Section 4.4. Subsequently, the one-dimensional column apparatus for the investigation of the geosynthetic material are described in Section 4.5. The apparatus for constructing two-dimensional capillary barrier model are described in Section 4.6. The experimental programmes for the research are given in Section 4.7. The numerical analysis methods adopted in this study are explained in Section 4.8. Finally, section 4.9 explains the programme adopted in the parametric study.

4.2 Outline of Research Programme

This research mainly focused on laboratory experiments. Nevertheless, numerical analyses and parametric study were also carried out. Numerical analysis of the infiltration and evaporation processes in capillary barrier model was conducted using saturated/unsaturated finite element seepage modeling software, SVFlux (Soilvision System Ltd., 2004).

The experiments conducted in this research are listed as follows:

1. Properties tests of the materials used in the study, include:
 - index properties tests
 - compaction tests
 - tests for the determination of drying and wetting soil-water characteristic curves
 - tests for the determination saturated coefficient of permeability

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2. Evaporation and shrinkage tests of the residual soil and soil mixtures under ambient conditions and using additional evaporation simulator. The soils investigated in this study are listed below:

- residual soil (RS)
- 80% residual soil + 20% fine sand (MRF_80-20)
- 60% residual soil + 40% fine sand (MRF_60-40)

The definitions of the soil's name are as follows:

- MRF = Mixture of Residual soil and Fine sand
- 80-20 = combination of 80% residual soil and 20% fine sand
- 60-40 = combination of 60% residual soil and 40% fine sand

3. Infiltration column tests for investigating the geosynthetic material.

- soil column model consists of a fine sand layer overlying a gravelly sand layer (SCM_FS-GS)
- soil column model consists of a fine sand layer overlying a geosynthetic material (SCM_FS-GE)

The definitions of the model's name are as follows:

- SCM = Soil Column Model
- FS = Fine Sand (fine-grained layer)
- GS = Gravelly Sand (coarse-grained layer)
- GE = Geosynthetic material (coarse-grained layer)

4. Tests on laboratory capillary barrier model for investigating the use of residual soil and geosynthetic material in capillary barrier system.

- capillary barrier model consist of a residual soil as the fine-grained layer and gravelly sand as the coarse-grained layer (CBM_RS-GS)
- capillary barrier model consist of a mixture of residual soil and fine sand as the fine-grained layer and gravelly sand as the coarse-grained layer (CBM_MRF-GS)
- capillary barrier model consist of a fine sand as the fine-grained layer and geosynthetic material as the coarse-grained layer (CBM_FS-GE)

The definitions of the model's name are as follows:

- CBM = Capillary Barrier Model
- RS = Residual soil (fine-grained layer)

- MRF = Mixture of Residual soil and Fine sand (fine-grained layer)
- FS = Fine Sand (fine-grained layer)
- GS = Gravelly Sand (coarse-grained layer)
- GE = Geosynthetic material (coarse-grained layer)

4.3 Investigation of Material Properties

This section describes the test methods to determine the index properties and water characteristic curve of the materials used in this study. The fitting of water characteristic curves and determination of permeability function are also discussed in this section.

4.3.1 Basic soil properties

The soil used in this study was taken from a construction site at Hoover Park, Singapore. It is a residual soil which is the product of weathering from Bukit Timah Granitic formation. Prior to index properties tests, the soil was oven dried and then ground. The basic property tests and standards for each test are summarized in Table 4.1.

Table 4.1 Basic soil properties tests and standards

NO	DESCRIPTION	STANDARD
1	Unified Soil Classification System	ASTM: D 2487 - 93
2	Specific gravity	ASTM: D 854 - 92
3	Grain size analysis	
	- Dry sieve analysis	ASTM: D 422 - 63
	- Wet sieve analysis	ASTM: D 1140 - 92
	- Hydrometer test	ASTM: D 422 - 63
4	Atterberg limits	
	- Liquid limit (Cone penetrometer)	BS 1377: Part 2: 1990-4.3
	- Liquid limit (Casagrande device)	ASTM: D 4318 - 95a
	- Plastic limit & Plasticity index	ASTM: D 4318 - 95a
5	Compaction	ASTM: D 698 - 91
6	Falling head saturated permeability	ASTM: D 5856 - 95

Determination of dry density and water content achieved in the capillary barrier model

In addition to the Standard Proctor Compaction test, a compaction test using a vibratory compactor was also carried out to obtain the achievable dry density of the capillary barrier model. The vibratory compactor used in the study was operated by air pressure. Hence, the applied compaction energy from the vibratory compactor was regulated by controlling the air pressure and also number of blows so that it does not exceed the strength of the infiltration box. Figure 4.1 shows the compaction test using the vibratory compactor.

From the vibration compaction test, it was observed that it would be difficult to compact the residual soil using the vibratory compactor if the water content was higher than the optimum water content obtained from the Standard Proctor compaction test (in this study the optimum water content was 24%). At water contents higher than 24%, a large number of soil lumps were formed, thus it was difficult to obtain a uniform soil layer. Therefore, a water content at the dry of optimum condition from the Standard Proctor compaction test was selected as the initial condition for the residual soil used in the experiments. A water content of 22% and a dry density of 1.15 Mg/m³ were selected to be used for the residual soil in the capillary barrier model.

Figure 4.1 Compaction test using vibratory compactor

4.3.2 Test methods to obtain water characteristic curve

This section describes the test methods to obtain the drying and wetting water characteristic curves for soil and geosynthetic material.

4.3.2.1 Drying soil-water characteristic curve

The drying soil-water characteristic curve (SWCC) was obtained by carrying out tests using a Tempe pressure cell and a Pressure plate extractor. The Tempe pressure cell was used to obtain SWCC up to a matric suction of 100 kPa. Subsequently, the test was continued using the Pressure plate extractor up to a matric suction of 500 kPa.

Tempe pressure cell manufactured by Soilmoisture Equipment Corporation, model No. 1405 B01M3-3, with 88 mm diameter and 30 mm height was used in this study (Figure 4.2). The Tempe pressure cell was equipped with 1-bar high air-entry ceramic disk.

In the Tempe pressure cell test, a matric suction was applied by maintaining a zero pore-water pressure and applying a positive pore-air pressure (i.e., axis-translation technique (Hilf, 1956)). Since the air pressure supplied through the pressure gauge could only be accurate in 10 kPa increment, while the air-entry value of residual soil is normally less than 10 kPa, a modification to the air pressure line was carried out. The schematic diagram of the modified air pressure line is shown in Figure 4.3. Water head was used to quantify the low air pressure applied to the Tempe cell. An accurate air pressure as low as 1 kPa could be obtained by adjusting the air pressure release valve.

Prior to the placement in the Tempe pressure cell, the residual soil specimen was mixed with water to a desired water content (in this study it was 22 %). Subsequently, the specimen was compacted to a target dry density of 1.15 Mg/m³. The test was started by saturating the soil specimen using a burette which was connected to the water outflow outlet below the ceramic disk. After saturating the specimen, the top plate was mounted and tightened. The top and bottom plates were sealed with O-rings to obtain an airtight condition during the test.

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After the specimen was saturated, air pressure was applied to the Tempe pressure cell. As the air pressure was applied, water started draining from the specimen through the high air-entry disk. The change in water content of the soil specimen was calculated by measuring the mass of the whole cell periodically until equilibrium condition was reached. To verify the equilibrium condition, the volume of water in the soil specimen was plotted against the logarithmic of the elapsed time during the test. The equilibrium condition was defined as a condition when there is no change in the volume of water in the soil specimen. This procedure was subsequently repeated at higher matric suctions. After the matric suction reached 100 kPa, the soil specimen was removed from Tempe pressure cell and then placed in the Pressure plate extractor (Figure 4.4).

(a) Disassembled components of a Tempe pressure cell

(b) Assembled Tempe pressure cell

Figure 4.2 Assemblage of the Tempe pressure cell (Soilmoisture Equipment Corporation, 1985; after Fredlund and Rahardjo, 1993a)

Figure 4.3 Schematic diagram of pressure line set-up for Tempe pressure cell test

Figure 4.4 5-Bar Pressure plate test

A Pressure plate extractor manufactured by Soilmoisture Equipment Corporation was used in this study. The procedure of pressure plate test is described in ASTM D 2325. The soil specimen was placed on top of a high air-entry ceramic disk and then air pressure was applied to attain the desired matric suction. The application of matric suction to the soil caused water to drain out to the water compartment through the ceramic disk. The ceramic disk does not allow air to flow through it as long as the applied matric suction does not exceed the air entry value of the ceramic disk. A 5-bar ceramic disk produced by Soilmoisture Equipment Corporation was used in this experiment. According to Fredlund and Rahardjo (1993a), the air-entry value of the disk is higher than 550 kPa. Hence, the maximum applied matric suction in the pressure plate test was set to 500 kPa. To calculate the water volume changes during the test, the chamber was dismantled and the mass of the soil specimen was measured. Similar to the Tempe pressure cell test, the volume of water in the soil specimen was plotted against the logarithmic of elapsed time to verify the equilibrium condition. This procedure was repeated for all matric suctions applied to the soil specimens. After equilibrium at the highest matric suction (500 kPa) was attained, the water content of the soil specimen was measured by oven-drying the specimen. This final water content was used to back-calculate the water contents corresponding to other matric suction values.

The drying SWCC was obtained by plotting the water contents at equilibrium condition against the corresponding matric suctions.

4.3.2.2 *Wetting soil-water characteristic curve*

The wetting SWCC was obtained using capillary rise test. A tube of 500 mm in height and 100 mm in diameter was used for the capillary rise test (Figure 4.5). Several holes were drilled along the tube at 50 mm spacing to collect specimens for water content determination. The residual soil was compacted at a target dry density (similar to the dry density on the Tempe cell test) and then the tube was placed in a tray with a constant water level at the bottom of the tube. The top of the tube was covered to prevent evaporation. The test was carried out until equilibrium of pore-water pressures were established throughout the soil specimen. Indrawan (2004) measured the pore-water pressure along the soil

specimen and found that water reached the upper part of the tube in less than 2 days. However, the tests were continued until approximately 20 days to obtain an equilibrium condition along the soil specimen. After the experiment stopped, soil samples were taken from different levels of the tube and then oven-dried to obtain the water contents. The elevation of the specimens above the water table is equal to the capillary head or negative pore-water pressure head, which can be used to calculate the matric suction. The measured water contents are plotted against the corresponding matric suctions to obtain the wetting SWCC.

Figure 4.5 Capillary rise tube set-up

4.3.2.3 Water characteristic curve of geosynthetic material

One of the objectives of this research is to investigate the possibility of replacing gravel layer in a capillary barrier system with a geosynthetic material. The geosynthetic material used in this study is Polyfelt Megadrain 2040 (Polyfelt Geosynthetics), which consists of a matrix of polypropylene monofilaments sandwiched between two layers of polypropylene geotextile (Figure 4.6; see Appendix A). It should be noted that water in the capillary barrier system is

designed to flow above the geotextile and not in the monofilament mat. Therefore, the water characteristic curve tests were conducted on the filter geotextile.

Figure 4.6 Cross section of Polyfelt Megadrain 2040 (Polyfelt Geosynthetics)

Both drying and wetting water characteristic curves of the geotextile (Geotextile-Water Characteristic Curve, GWCC) were obtained using capillary rise principle. Geotextile specimens of 300 mm in length and 50 mm in width were used in the GWCC test. About 20 mm of the length was submerged in a water container and the remaining 280 mm was hung above the water level which was maintained constant during the test as illustrated in Figure 4.7. For drying GWCC test, the geotextile specimen was saturated prior to the test, while for wetting GWCC test, an initially dry geotextile specimen was used for the test. The tests were stopped when the equilibrium conditions were reached.

Figure 4.7 Set-up of geosynthetic capillary rise test

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As it was difficult to visually observe the equilibrium condition, the geosynthetic capillary rise tests were conducted by limiting the duration of the tests. Several tests were conducted at different durations (1, 2, 3, and 4 days) and the water contents of the geotextile were measured at the end of each test. The geotextile specimens were cut at every 5 mm and then subsequently were oven-dried to obtain the water contents. The drying and wetting GWCC were obtained by plotting the water contents of each specimen from different heights against the corresponding matric suctions, which were determined based on the height of the specimen above the water table.

4.3.2.4 Fitting of water characteristic curve

To obtain a smooth and full-range water characteristic curves, the volumetric water content and the matric suction data obtained from the tests were fitted using Fredlund and Xing (1994) equation. The water characteristic curves of the materials used in this study and the best-fitting parameters are presented in Chapter 5.

4.3.3 Determination of permeability functions

The permeability function of the soils used in this study was calculated using the statistical method presented in Fredlund and Rahardjo (1993a). The permeability function of the fine-grained layers in the capillary barrier models were also calculated using the instantaneous profile method (Watson, 1966; Hillel et al., 1972 and Olson and Daniel, 1981; after Fredlund and Rahardjo, 1993a). In this study, the instantaneous profile method was used to obtain the permeability function during the drying process. During the drying process, water was assumed to flow upwards from the bottom of the fine-grained layers. Pore-water pressure data from tensiometers during drying processes of several tests in each capillary barrier model were used to calculate the permeability functions. The statistical method and instantaneous profile method are explained in details in Section 3.7. The permeability functions of the soils used in the study are presented in Chapter 5.

4.4 Evaporation and Shrinkage Tests

The evaporation and shrinkage characteristics of the residual soils were studied using cylindrical soil specimens of 103 mm in diameter and 50 mm in height. A base plate with holes, a polyvinyl chloride (PVC) cylinder and a soil compactor were used to prepare the specimen (Figure 4.8). The soil specimens were compacted at a specified initial water content and dry density. Prior to the compaction of the specimen, a filter paper was placed at the bottom of the PVC cylinder to prevent the migration of soil particles through the holes of the base plate. The specimen was then compacted layer by layer as illustrated in Figure 4.9. After compaction had been completed, the soil specimen was saturated by placing the entire system into a container of water. An additional PVC cylinder for saturating the soil from the top was installed above the specimen as illustrated in Figure 4.10. Once the soil specimen was saturated, it was carefully extruded from the mould and then placed on an acrylic tray. Subsequently, the specimen was placed in an acrylic box and the selected drying condition was applied to the specimen.

Three different drying conditions of evaporation and shrinkage tests were investigated. The specimens were dried under an ambient condition, under an accelerated evaporation using lamp and under an accelerated evaporation using fan. The set-up of the evaporation and shrinkage test with the lamp as an additional evaporation simulator is illustrated in Figure 4.11.

Figure 4.8 Equipment used in the evaporation and shrinkage tests

Figure 4.9 Preparation of the evaporation and shrinkage tests specimen

Figure 4.10 Saturation process of the evaporation and shrinkage tests specimen

The mass and dimension of the specimens were measured periodically to obtain the evaporation and shrinkage characteristics of the soils. The mass of the specimen was used to calculate the change in water content and also the actual evaporation from the specimen. The height and diameter of the specimens were used to calculate the change in void ratio of the specimens. Figure 4.12 illustrates the dimension measurement of the specimen using a vernier caliper.

Other than the actual evaporation from the specimen, potential evaporation under the same drying conditions was also measured. Potential evaporation is defined as the amount of water which is evaporated from a free water surface. It was

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measured simultaneously during the tests by measuring the changes in the mass of water in a plastic beaker under the same drying conditions. The potential evaporation during the tests were measured to provide a comparison with the potential evaporation observed in the field. Air relative humidity and air temperature during the tests were also measured periodically using a portable thermo-hygrometer.

Figure 4.11 Set-up of the evaporation and shrinkage test with evaporation simulator

Figure 4.12 Measurement of the evaporation and shrinkage test specimen

The evaporation and shrinkage tests were carried out until there was no significant change in the weight of the specimens (water content of the specimen approaching 0%).

4.5 Column for Studying Infiltration through Geosynthetic Material

This section describes the apparatus and methods for constructing the soil column model for the investigation of the geosynthetic material. The apparatus consists of three main components, i.e. the acrylic cylinder, measuring devices, and water-flow system. The schematic diagram and the photograph of the infiltration column apparatus are presented in Figures 4.13 and 4.14, respectively.

4.5.1 *The acrylic cylinder*

The cylinder used in the study was made of acrylic transparent tube with a 5mm thick wall, 190mm internal diameter and 1000mm height. A number of holes for installing measuring devices were drilled along the cylinder. The cylinder was fixed on a stainless steel base plate, which was supported with four steel rods that were fixed on the concrete floor. To prevent water leakage, the joints between the cylinder and the base plate were sealed using rubber O-ring and fastened with bolts and nuts.

4.5.2 *Measuring devices and data acquisition system*

A number of measuring devices were installed in the soil column model. Small tip tensiometers and time domain reflectometry (TDR) were installed along the column to measure negative pore-water pressure and volumetric water content, respectively. The tensiometers were attached to pressure transducers, which were then connected to a data acquisition system to obtain continuous measurements. To obtain a continuous measurement of volumetric water content, the TDRs' were connected to a Trase system which was controlled using a personal computer. Other than the pore-water pressure and volumetric water content, overflowed and percolated water were also measured during the tests. Electronic weighing balances with resolutions of 1 g were used for these purposes. More details on the measuring devices used in the infiltration column are explained below.

Figure 4.13 Schematic diagram of the infiltration column apparatus (Yang et al., 2004)

Figure 4.14 Photograph of the infiltration column apparatus

4.5.2.1 Tensiometer - transducer system

Tensiometer is a device for direct measurement of negative pore-water pressure. Tensiometer model 2100F (Soilmoisture Equipment Corporation, CA, USA) equipped with a jet fill water reservoir is used in this study (Figure 4.15). The tensiometer consists of a 1-bar air entry porous ceramic cup, a tensiometer tube, and a jet fill water reservoir. The porous ceramic cup is a small tip which has a diameter of 6 mm and a length of 25 mm and attached to the tensiometer body via a polyethylene tube. The jet fill system was used to remove the accumulated air bubbles in the tensiometer tube. The mechanism of the jet fill is similar to the action of a vacuum pump. When the jet fill button is pressed, water is injected to the tube and the air bubbles move upward to the water reservoir.

Figure 4.15 Small tip tensiometer equipped with jet-fill water reservoir (schematic diagram from Tami, 2004)

A good contact between the ceramic cup and the soil is necessary to establish continuity between the pore-water in the soil and the water in the tensiometer tube. Therefore, the ceramic cup was installed during the compaction of the soil. It should be noted that the compaction of the soil above the ceramic cup should be done carefully to avoid any damage to the ceramic cup or blockage in its coaxial tubing. When the ceramic cup is in contact with soil, the water in the tensiometer

will have the same negative pressure as the pore-water in the soil. This negative pore-water pressure is measured by a pressure transducer. The negative pore-water pressure that can be measured in a tensiometer is limited to approximately -90 kPa due to cavitation of water in the tensiometer (Fredlund and Rahardjo, 1993a). When the pore-air pressure is atmospheric, the measured negative pore-water pressure is numerically equal to the matric suction. The matric suction measured should not exceed the air-entry value of the ceramic cup. A ceramic cup with an air-entry value of around 100 kPa was used in this study. The use of ceramic cup with an air-entry value greater than 100 kPa will not increase the measuring range of the tensiometer since water in the tube will always cavitate when the water pressure is approximately -90 kPa.

To obtain a continuous measurement of negative pore-water pressure, the tensiometers were connected to pressure transducers and subsequently connected to a data acquisition system. The pressure transducers used in this study were Kristal K-line Current Pressure Transducers, type RAN25A2BV0215, manufactured by Kistler® (Kristal Instrumente AG, Winterthur, Switzerland) with a measuring range from -100 to 75 kPa.

Before attaching the transducers into the tensiometer tube, the transducers and the data acquisition system were calibrated using water pressure head. At least six water pressure heads were applied to obtain the relationship between water pressure head and the output voltage from the transducer. The water pressure head versus output voltage was plotted and fitted with a linear regression line to obtain the equation of correlation (Figure 4.16). This equation was then used in the data acquisition software to convert the output voltage into water pressure head automatically.

4.5.2.2 Data acquisition system

In the infiltration column setup, two data logger were used to obtain continuous readings of pore-water pressure measurement from the tensiometer-transducer system. The data loggers are DataShuttle™ brand, type DS-16-8-GP, manufactured by IOtech Incorporation (Figure 4.17). Each data logger has 8 channels and allows 8 pressure transducers to be connected. The data loggers were connected to a

personal computer through a parallel port. DASYLab® software was used to control the measurement setting of the data logger.

Figure 4.16 Linear relationship between voltage and pressure head in the transducer

Figure 4.17 Data acquisition system and Trase system

4.5.2.3 Time domain reflectometry (TDR) and Trase system

Time domain reflectometry was used to determine the volumetric water content of the soil column. The TDR system used in this study is manufactured by Soilmoisture Equipment Corporation, CA, USA. The system includes a Trase BE 6050X2, a 76-channel Multiplexer Enclosure 6020B05, a 16-channel TDR Switching board 6021C16, a 4-MB Memory Card 6058C10, and 8 cm Buriable Waveguides. In addition, Wintrase Kit 6300K2-BE software was used to control the system and record the data. The Trase system used in this study is shown in Figure 4.17 and the TDR waveguides are shown in Figure 4.18.

Figure 4.18 TDR waveguides used in this study

A typical TDR system consists of three main parts: a step pulse generator, coaxial cables, and waveguides. The step pulse generator generates an electromagnetic wave of energy that travels to the waveguide with a certain speed. The speed of the wave depends on the dielectric constant, K_a , of the material in contact with the waveguide. The higher the dielectric constant, the lower the speed of the wave. In general, soil is composed of air, mineral particles, and water. The dielectric constants, K_a , for these materials are 1, 2 to 4, and 80 for air, mineral particles, and water, respectively. Because of the large difference in the dielectric constant of water and the other components in a soil, the travel speed of the wave in a parallel

transmission buried in the soil is mainly dependent on the water content of the soil.

The “apparent” dielectric constant, K_a , of the air-water-soil system is determined by the following formula.

$$K_a = \left(\frac{t \cdot c}{L} \right)^2 \quad (4.1)$$

Where:

K_a = dielectric constant

t = transit time (nanoseconds), defined as the time required for the pulse to travel in one direction from the start to the end of the waveguide

c = speed of light (cm/nanosecond)

L = length of the waveguide (cm)

The relationship of the K_a value with the volumetric water content of soil had been established as shown in the Figure 4.19.

Figure 4.19 Relationship of dielectric constant, K_a , to volumetric water content of soil, θ_w (Trase system operating instruction, Soilmoisture Equipment Corp.)

4.5.2.4 Electronic weighing balance

Two electronic weighing balances were used to obtain the overflowed and percolated water in the infiltration column tests. Electronic weighing balance manufactured by UWE model PS-12K was used in this study (Figure 4.20). The capacity of each weighing balance is 12 kg with resolution of 1 g. A digital indicator manufactured by Kubota model KL-D1000 was connected to each weighing balance. The digital indicator displayed the value measured by the weighing balance. To obtain continuous measurements, the digital indicator was connected to a personal computer through an RS-232 port. The digital indicator was controlled using a self-created interface program that was able to set up the intervals and duration of the data acquisition.

Figure 4.20 Electronic weighing balance and digital indicator

The electronic weighing balances used in this study were calibrated using one set of standard metric weights (1 to 5000 g) Model No. TW-5000-1, manufactured by Troemner, PA, USA. The result of calibration indicated that the accuracy of the electronic weighing balances were up to 1 g.

4.5.3 Water-flow system

There are three components in the water flow system of the infiltration column, i.e. inflow control, overflow discharge and percolation discharge.

The inflow control consisted of a water storage tank, a constant head water tank, a flow regulator, and a rainfall distributor. The water storage tank provided a continuous supply of water to the constant head water tank. Subsequently, water flowed from the constant head water tank into the rainfall distributor through a flow regulator. The rainfall distributor was made of a plastic nozzle with a small hole. To obtain different flow rates, several nozzles with different size of holes were fabricated. A desired water flow rate was obtained by using a specific nozzle and also by adjusting the flow regulator. During the calibration of the nozzle, it was found out that the water flow rate decreased with time. This problem might be caused due to fine particles or air bubbles developed at the nozzles during the rainfall tests. Hence, the flow rate was measured before and after rainfall tests and the applied rainfall intensity would be the average value. To obtain a uniform distribution of rainfall in the soil column, a filter paper was placed in contact with the surface of the soil. The uniformity was verified by installing two tensiometer tips at the same elevation near the surface. The tensiometer tips were installed at the center and at the edge of the column. Results of rainfall tests showed that both tensiometers measured a similar pore-water pressure and also changed at the same time during rainfall.

The second component of the water flow system is the overflow discharge system. Two outlets were provided at the same level as the soil surface to discharge the overflow water during rainfall tests.

The last component of the water flow system, which is the percolation discharge system, was provided using an outlet on the base plate at the bottom of the soil column. Another outlet which is located on the other side of the base plate was connected to a water tank that was filled with water to maintain the water table at the bottom of the column.

4.5.4 Placement of soil and installation of measuring devices of the soil column

To achieve a homogeneous soil layer in a soil column model, the soil was compacted in layers of around 100 mm thickness. A 2.5 kg Proctor hammer was dropped 25 times from a height of 30 cm on each layer. Measuring devices, such as tensiometer tip and TDR waveguide, were installed during the compaction to obtain a good contact between the devices and the soil (Figure 4.21). To prevent leakage, the measuring devices were installed using connectors and sealed with rubber o-rings. There were 10 tensiometer tips installed at 9 different elevations (two tensiometer tips were installed at the top to investigate the uniformity of rainfall) and 8 TDR waveguides in the soil column. The arrangement of the TDR waveguides and tensiometer tips is shown in Figure 4.22.

Figure 4.21 Installation of measuring devices in the soil column

4.6 Apparatus for Constructing Capillary Barrier Model

The detail of the apparatus used for constructing capillary barrier model is described in this section. The apparatus consists of five main components, i.e. infiltration box, measuring devices, water-flow system, rainfall simulator, and evaporation simulator.

Figure 4.22 Arrangement of measuring devices in the soil column

4.6.1 Infiltration box

An infiltration box of 2.45 m in length, 2.00 m in height, and 0.40 m in width was fabricated using a stainless steel frame and acrylic sheets as the sidewalls (Figure 4.23). Acrylic sheets were used in order to provide easier installation of measuring devices, placement of soil, and visual observation of the capillary barrier performance. Watertightness was a priority in fabricating the infiltration box. Therefore, all joints were sealed with silicone rubber to prevent leakage.

A number of holes for installing measuring device connectors were drilled along the sidewalls. Several additional holes for collection of run-off and drainage were also provided at the front panel of the box. A PVC plate was installed as the base of the capillary barrier model to form an impervious boundary condition during the experiments.

4.6.2 Measuring devices and data acquisition system

Similar to the soil column, a number of measuring devices were also installed in the capillary barrier model. Tensiometers and time domain reflectometry were installed along the capillary barrier model to measure negative pore-water

pressure and volumetric water content, respectively. To measure the amount of the applied rainfall, a digital flowmeter was installed in the system. Three electronic weighing balances were used to measure the amount of run-off, lateral drainage, and percolation. All the measuring devices were connected to a data acquisition system to obtain continuous measurements. The tensiometer-transducer system that was used in the capillary barrier model was similar to the one used in the soil column. The time domain reflectometry and Trase system was also the same with the one used in the soil column, except that the length of the TDR waveguide was 20 cm instead of 8 cm (Figure 4.18). This section describes the measuring devices that have not described in Section 4.5.2.

Figure 4.23 Experimental set-up of capillary barrier model (GS: gravelly sand layer, RS: residual soil layer, RF: rainfall simulator, EV 1: lamp, EV 2: fan, WB: weighing balance, TDR: multiplexer for time domain reflectometry, DAS: data acquisition system)

4.6.2.1 Electromagnetic flowmeter

An electromagnetic flowmeter was installed in the water supply system of rainfall simulator to measure the amount of water flowing into the simulator. An electromagnetic flowmeter manufactured by Danfoss Instrument, namely Magflow® (Figure 4.24), was selected because of its accuracy and suitability for measuring the amount of water flow to the rainfall simulator. The flowmeter consists of a flow sensor and a signal converter. The flow sensor model MAG 1100 and the signal converter model MAG 5000 were used in the flowmeter. The flow sensor MAG 1100 of DN6 size is able to measure the water flow from 25 mm/hr to 1000 mm/hr with 5% accuracy. The signal converter MAG 5000 was connected to the data acquisition system to obtain continuous measurements during the test.

Figure 4.24 Electromagnetic flowmeter

4.6.3 Water-flow system

Figure 4.25 shows the water-flow system of the capillary barrier model. A 350L sump tank located below the capillary barrier model was used to store the water for the rainfall simulator. A submersible centrifuge pump with a maximum flow capacity of 10 L/min was used to pump the water into the rainfall simulator.

As illustrated in Figure 4.25, the simulated rainfall applied to the capillary barrier model turned into run-off and infiltration. The infiltrated water was diverted laterally in the fine-grained layer and also percolated into the coarse-grained layer, if it did occur. The run-off from the capillary barrier model was measured from the output drainage which came from the water flow on the surface of the capillary

barrier model. The lateral diversion was measured from the amount of the output drainage generated by the fine-grained layer of the capillary barrier model, while the percolation was measured from the amount of the output drainage generated by the coarse-grained layer of the capillary barrier model. Auto-discharge tanks (Figure 4.26) were used to collect these output drainage from the capillary barrier model. The amount of water collected in each auto-discharge tank was measured by the electronic weighing balance.

Figure 4.25 Drainage system of the capillary barrier model (modified from Tami, 2004)

Figure 4.26 Auto discharge tank equipped with electronic weighing balance

4.6.4 Rainfall simulator

A rainfall simulator was designed to simulate rainfall in the capillary barrier model. Two nozzles were mounted at both ends of the infiltration box to supply water. High air pressure was used to distribute water mist throughout the capillary barrier model. Intensity of the rainfall was controlled by the size of the nozzle, while the uniformity of the rainfall was controlled by the position of the nozzle and the air pressure applied to the system. The arrangement of rainfall simulator in the capillary barrier model is illustrated in Figure 4.25.

Calibration of the rainfall simulator

The uniformity of rainfall applied to the capillary barrier model was calibrated prior to the experiments. Ten measuring cylinders were distributed along the capillary barrier model (Figure 4.27). Two rainfall intensities, 10 mm/hr and 15 mm/hr for one hour were used for the calibration purpose. The Christiansen Uniformity Coefficient described by Hall et al. (1989) was used to quantify the uniformity of the rainfall applied.

Figure 4.27 Measuring cylinder arrangement on the calibration of the rainfall simulator

$$\text{Christiansen Uniformity Coefficient} = 100 \times \left(1 - \frac{\sum_{i=1}^n |X_i - \bar{X}|}{\sum_{i=1}^n X_i} \right) \quad (4.2)$$

Where:

X_i = Observed data

$\bar{X}$ = Mean of the observed data

Two factors were investigated in order to obtain a uniform rainfall distribution. The air pressure was varied in the range of 250 kPa to 350 kPa and the nozzles were adjusted at several positions until the maximum uniformity coefficient was achieved. The uniformity coefficients calculated from the calibration data were varied from 60.11% to 84.60% with an average of 72.82%.

Other than the uniformity of the rainfall, the average value of the rainfall intensity also had to be considered in the calibration process. The calculation of the confidence interval on μ (average value of rainfall intensity) for an applied rainfall of 15 mm/hr under 300 kPa of air pressure is presented below.

Height of rainfall observed in the measuring cylinder (in mm):

9 14.5 16.5 17 15 17 16.5 15 12 11

The mean of the above data is $\bar{x} = 14.35$ with a standard deviation, $s = 2.78$.

The 95% confidence interval on μ (average value) with an unknown variance is calculated using the t-distribution as follows:

$$\bar{x} - t_{\alpha/2, n-1} \cdot s / \sqrt{n} \leq \mu \leq \bar{x} + t_{\alpha/2, n-1} \cdot s / \sqrt{n}$$

$$14.35 - t_{0.025, 9} \cdot 2.78 / \sqrt{10} \leq \mu \leq 14.35 + t_{0.025, 9} \cdot 2.78 / \sqrt{10}$$

$$14.35 - 2.262 \cdot 2.78 / \sqrt{10} \leq \mu \leq 14.35 + 2.262 \cdot 2.78 / \sqrt{10}$$

$$12.36 \leq \mu \leq 16.34$$

From this result, it can be concluded that for an applied rainfall of 15 mm/hr, the average value of the rainfall distributed on the capillary barrier model was in the range of 12.36 mm/hr to 16.34 mm/hr.

The results of the uniformity coefficient and the 95% confidence interval on the average value calculations for other applied rainfalls and air pressures are summarized in Table 4.2.

Table 4.2 Summary of rainfall calibration’s calculation results

APPLIED RAINFALL (mm/h)	APPLIED AIR PRESSURE (kPa)	AVERAGE OF THE MEASURED RAINFALL (mm/h)	UNIFORMITY COEFFICIENT (%)	95% CONFIDENCE INTERVAL ON μ
10	250	12.35	60.11	8.28 - 16.42
	300	9.15	76.60	5.88 - 12.43
	350	7.95	64.78	6.44 - 9.46
15	250	18.10	70.83	12.89 - 23.31
	300	14.35	84.60	12.36 - 16.34
	350	10.75	80.00	8.80 - 12.70

Based on the calibration results, it was decided that an air pressure of 300 kPa would be used for the entire experiment because it gave the highest uniformity coefficient and a good range of average rainfall intensities to the capillary barrier model.

4.6.5 Evaporation simulator

As it is known that evaporation rate is governed by atmospheric conditions such as relative humidity, air temperature, and wind speed, the evaporation simulator was designed to simulate the changes in these conditions. Two equipments were used as the evaporation simulator. First, three infrared light bulbs with wattage of 175 w, were used to simulate net radiation and air temperature. Using steel frames, the light bulbs were installed in the infiltration box above the capillary barrier model at a distance of about 0.5 m above the soil surface. To control the air temperature above the soil, the intensity of the light bulbs was controlled using a light dimmer. Secondly, to simulate wind in the field, three fans were installed at both ends of the infiltration box using specially designed steel frames. The air flow capacity of each fan was 84 L/s. The arrangement of the evaporation simulator is illustrated in Figures 4.23 and 4.25.

4.6.6 Placement of soil and installation of measuring devices for the capillary barrier model

To achieve a homogeneous soil layer in a capillary barrier model, the soil was compacted in layers of 20 to 30 mm thickness. A vibratory compactor with a 150 mm by 150 mm plate was used to compact the soil layer. As explained in Section 4.3.1, the energy applied to compact the soil in the infiltration box was regulated by controlling the air pressure and number of blows.

Measuring devices, such as tensiometer tip and TDR waveguides, were installed along the soil column model and the capillary barrier model. To obtain a good contact between the device and the soil, the installation was conducted during the placement of the soil. To prevent leakage in the infiltration box, the measuring

devices were installed using connectors and sealed with rubber O-rings. Figure 4.28 shows the TDR waveguide and tensiometer tip with their connectors.

Figure 4.28 TDR waveguide and tensiometer tip with their connectors

A total of 14 TDR waveguides and 20 tensiometer tips were installed in the capillary barrier model. The arrangement of the TDR waveguides and tensiometer tips in the capillary barrier model is shown in Figure 4.29.

Figure 4.29 Arrangement of measuring devices in the capillary barrier model

4.7 Experimental Programme

The detail experimental programme for the evaporation and shrinkage tests, soil column models, and capillary barrier models are explained in this section.

The soil column models and the capillary barrier models were subjected to infiltration processes. The rainfall intensities applied to the models were based on the observation of past slope failures in NTU. According to Rahardjo et al. (2001), a five day antecedent rainfall exceeding 60 mm combined with a daily rainfall greater than 90 mm appears to be sufficient to cause landslides. Hence, the models were subjected to rainfall intensities based on these values.

4.7.1 Experimental programme for evaporation and shrinkage tests

To reduce the shrinkage potential and improve the hydraulic properties, the residual soil was mixed with fine sand in different proportions by dry mass. The soils that were investigated in this study are as follows:

- residual soil (RS)
- 80% residual soil + 20% fine sand (MRF_80-20)
- 60% residual soil + 40% fine sand (MRF_60-40)

Evaporation and shrinkage tests under three different drying conditions were carried out for all the soils. The tests were conducted under an ambient condition, an accelerated evaporation using lamp, and an accelerated evaporation using fan. The soils were then evaluated based on their index properties, volume change characteristics, and hydraulic properties. The selected soil mixture (MRF_60-40) was then used as fine-grained layer in the laboratory capillary barrier model.

4.7.2 Experimental programme for soil column models

Two soil column models were investigated in this study. The first soil column model was constructed using 0.45 m thick fine sand as the fine-grained layer overlying 0.08 m thick gravelly sand as the coarse-grained layer (SCM_FS-GS). The second soil column model was constructed using 0.51 m thick fine sand as the fine-grained layer overlying Polyfelt Megadrain 2040 as the coarse-grained layer

(SCM_FS-GE). A fine sand layer was placed below the gravelly sand and the geosynthetic material to represent the original soil which is protected by the capillary barrier. The thicknesses of the fine-grained layers in SCM_FS-GS and SCM_FS-GE were different in order to accommodate the location of the measuring devices. Schematic diagram of the soil column models are presented in Fig. 4.30.

Figure 4.30 Schematic diagram of (a) SCM_FS-GS and (b) SCM_FS-GE

Two type of tests were applied to each soil column model. The first test was a rapid drawdown test, which was conducted by lowering the water table from the top of the soil column ($z = 1.0\text{ m}$) to the bottom of the soil column ($z = 0$). The drawdown test was conducted to investigate the development of negative pore-water pressure in a drying process starting from a saturated condition. The equilibrium condition achieved at the end of the drawdown test was used as the initial condition for the second type of test which is the rainfall test. The tests conducted on the soil column models are listed in Table 4.3.

Table 4.3 Experimental programme for the soil column models

MODEL	TEST	INFILTRATION	
		Intensity (mm/hr)	Duration (hr)
1	SCM_FS-GS_DD	-	-
	SCM_FS-GS_RF1	5.8	6
	SCM_FS-GS_RF2	10.7	6
	SCM_FS-GS_RF3	4.9	9
2	SCM_FS-GE_DD	-	-
	SCM_FS-GE_RF1	5.8	6
	SCM_FS-GE_RF2	12.6	6
	SCM_FS-GE_RF3	4.3	9
DD = Rapid drawdown test RF = Rainfall test			

4.7.3 Experimental programme for capillary barrier models

Three capillary barrier models were investigated in this study. The first capillary barrier model was constructed using 0.2 m thick residual soil overlying 0.2 m thick gravelly sand (CBM_RS-GS). The second combination consists of 0.2 m of the mixture of residual soil and fine sand (MRF_60-40) overlying 0.2 m thick gravelly sand (CBM_MRF-GS). The last capillary barrier model was constructed using 0.2 m of fine sand overlying a geosynthetic material (CBM_FS-GE).

The capillary barrier models were subjected to infiltration and evaporation processes. The rainfall intensities applied to the capillary barrier model were based on the rainfall condition that caused slope failures in residual soil slope in NTU (Rahardjo et al., 2001). The infiltration process was then followed with an evaporation process. Using Penman’s equation, Gasmo (1997) calculated the potential evaporation rate, PE, for Singapore based on the data presented by the Meteorological Service of Singapore. The range of the potential evaporation for Singapore is from 5.16 to 7.53 mm/day or equal to 0.22 to 0.31 mm/hr. These values were used as an approximate potential evaporation for the capillary barrier models. The potential evaporation during the tests were measured by measuring the changes in the mass of water in plastic beakers under the same drying

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conditions. Three plastic beakers were placed at the bottom, middle, and top of the capillary barrier model for the potential evaporation measurement.

The tests on CBM_RS-GS were conducted to study the performance of a capillary barrier system constructed using residual soil as a fine-grained layer. The effects of different evaporation rates on the recovery of water storage were investigated. In addition, the effect of volume change behaviour on the long term performance of a capillary barrier system was also studied.

The second capillary barrier model, CBM_MRF-GS, was constructed to investigate the performance of a capillary barrier system using the mixture of residual soil and fine sand (MRF_60-40) as the fine-grained layer. The measurement of pore-water pressures, volumetric water contents, water balances, and the water storage capacity were used to analyze the performance of CBM_MRF-GS.

The use of a geosynthetic material as a coarse-grained layer in capillary barrier system was studied using the CBM_FS-GE. The performance of the capillary barrier using the geosynthetic material was examined based on the water balance, pore-water pressure, and water content measurements.

The experimental programme for the capillary barrier model is summarized in Table 4.4.

4.8 Numerical Analyses of the Soil Column and Capillary Barrier Models

Numerical analyses were conducted on both soil column and capillary barrier models. To verify the numerical model, the numerical analyses results were compared with the experimental results. Results of the numerical analyses are presented in Section 5.6.

Table 4.4 Experimental programme for the capillary barrier models

NO	NAME OF MODEL	NAME OF TEST	DESCRIPTION OF TEST	INFILTRATION			EVAPORATION			
				STAGING	INTENSITY (mm/hr)	DURATION (mm/hr)	MODE OF EVAPORATION	MEASURED POTENTIAL EVAPORATION (mm/hr)	DURATION (hr)	
1	CBM_RS-GS	CBM_RS-GS_RF1	Single event rainfall + evaporation	-	-	8.95	3	ambient condition	0.023	475
		CBM_RS-GS_RF2	Single event rainfall + evaporation	-	-	9.01	3	lamp	0.203	120
		CBM_RS-GS_RF3	Single event rainfall + evaporation	-	-	8.88	3	fan	0.120	120
		CBM_RS-GS_RF4	Single event rainfall + evaporation	-	-	9.04	3	lamp and fan	0.217	80
2	CBM_MRF-GS	CBM_MRF-GS_RF1	Single event rainfall + evaporation	-	-	10.26	3	lamp	0.214	120
		CBM_MRF-GS_RF2	Single event rainfall + evaporation	-	-	9.68	3	lamp and fan	0.426	100
		CBM_MRF-GS_RF3	Single event rainfall + evaporation	-	-	10.09	3	ambient condition	0.019	120
		CBM_MRF-GS_RF4	Single event rainfall + evaporation	-	-	9.88	3	lamp	0.186	120
		CBM_MRF-GS_RF5	Single event rainfall + evaporation	-	-	9.85	6	lamp and fan	0.521	120
		CBM_MRF-GS_RF6	Single event rainfall + evaporation	-	-	9.88	6	fan	0.239	120
		CBM_MRF-GS_RF7	Single event rainfall + evaporation	-	-	14.74	6	ambient condition	0.020	48
3	CBM_FS-GE	CBM_FS-GE_RF1	Rainfall test comprising of 3 stages	Stage 1	Adsorption	2.65	18	-	-	-
					No rainfall	0	24	-	-	-
				Stage 2	Adsorption	13.48	6	-	-	-
					No rainfall	0	0	-	-	-
	CBM_FS-GE_RF2	Single event rainfall	Stage 3	Desorption	3.07	18	-	-	-	
				No rainfall	0	24	-	-	-	
				Adsorption	10.07	6	-	-	-	
			-	No rainfall	0	39	-	-	-	
	CBM_FS-GE_RF3	Rainfall test comprising of 5 stages	Initial condition (SS)		1.62	36	-	-	-	
			Stage 1	Adsorption	8.12	5	-	-	-	
				No rainfall	0	20	-	-	-	
			Stage 2	Adsorption	7.87	5	-	-	-	
				No rainfall	0	20	-	-	-	
			Stage 3	Adsorption	20.41	5	-	-	-	
				No rainfall	0	22	-	-	-	
			Stage 4	Adsorption	7.85	5	-	-	-	
	CBM_FS-GE_RF4	Rainfall test comprising of 2 stages	Stage 5	Adsorption	7.86	10	-	-	-	
				No rainfall	0	14	-	-	-	
			Initial condition (SS)		4.02	24	-	-	-	
			Stage 1	Adsorption	14.82	6	-	-	-	
	CBM_FS-GE_RF5	Single event rainfall	Stage 2	Adsorption	14.68	6	-	-	-	
				No rainfall	0	40	-	-	-	
	CBM_FS-GE_RF6	Single event rainfall	-	Adsorption	9.83	3	-	-	-	
				No rainfall	0	45	-	-	-	
		CBM_FS-GE_RF6	Single event rainfall	-	Adsorption	31.07	3	-	-	-
				-	No rainfall	0	45	-	-	-

4.8.1 Softwares used in the numerical analysis

Numerical analyses of the infiltration and evaporation processes in the soil column and capillary barrier models were conducted using the saturated/unsaturated finite element seepage modeling software, SVFlux (Soilvision System Ltd., 2004). The infiltration process in the SVFlux is based on the governing equation for an unsteady state water seepage in unsaturated soil (Eq. 3.9). The evaporation process is based on the governing equation for water and water vapour flows under a non-isothermal condition (Eq. 3.10). The finite element model defined by the SVFlux was solved using a finite element solver package, namely FlexPDE (PDE Solutions Inc., 2002). The result of the numerical analysis was examined using a plotting software TecPlot (Amtec Engineering Inc., 2002).

4.8.2 Geometry and boundary conditions of the numerical models

The geometry of the numerical models was similar to the geometry of the laboratory models. The boundary conditions of the numerical models were also selected based on the conditions of the laboratory models.

In the soil column models, zero-flux boundary conditions were used at the side walls to simulate an impervious condition. During drawdown tests, the boundary conditions at the top of the soil columns were zero-flux, while the boundary conditions at the bottom of the soil columns were functions of the total head versus time obtained from the laboratory drawdown tests (see Appendix B). During rainfall tests, the boundary conditions at the top of the soil columns were flux boundary (the rainfall intensities applied to the model), while the boundary conditions at the bottom of the soil columns were zero pressure boundary.

Three types of boundary conditions were used in the numerical analysis of the sloping capillary barrier model. Flux boundary condition was used in the surface of the capillary barrier model. Zero-flux boundary condition was used at the bottom of the capillary barrier model to simulate an impervious condition. Zero pressure boundary condition was used to simulate the water table at the toe of the

capillary barrier model. The geometry and boundary conditions of the numerical model is presented in Figure 4.31.

Figure 4.31 Geometry and boundary condition of the numerical model

4.8.3 Material properties used in the numerical analysis

The most important properties needed for the numerical analysis of water flow in unsaturated soil are soil-water characteristic curve (SWCC) and permeability function. The SWCC of the soils were obtained from independent laboratory tests using the Tempe pressure cell and Pressure plate apparatuses for drying SWCC and capillary rise test for wetting SWCC (Section 4.3.2). The permeability function was calculated using the statistical method presented in Fredlund and Rahardjo (1993a) as explained in Section 3.7. The saturated coefficient of permeability obtained from independent laboratory test (the falling head test) and the SWCC were used to generate the permeability function.

The results of numerical analyses on CBM_RS-GS indicated that the saturated coefficient of permeability measured independently in the laboratory was lower than the saturated coefficient of permeability of the residual soil used in the model

(see Section 5.6.2.1). Therefore, a method to determine the permeability function of the soil in the capillary barrier model was adopted. Instantaneous profile method was selected due to the availability of pore-water pressure data along the capillary barrier models (Section 3.7). The numerical analyses of CBM_RS-GS and CBM_MRF-GS were then conducted using the permeability function obtained from the instantaneous profile method.

As suggested by Tami et al. (2004a), in the numerical analysis, appropriate hydraulic properties should be used in accordance with the process that the soils actually experience. Therefore, during infiltration process, wetting SWCC and wetting permeability function were used in the numerical analysis, while during evaporation process, drying SWCC and drying permeability function were used in the numerical analysis.

4.9 Parametric Studies

Using the verified numerical model, parametric studies were conducted to further study the use of residual soil as a fine-grained layer in capillary barrier.

4.9.1 Comparison of capillary barriers using different fine-grained layers

Three capillary barriers were analyzed to study the effect of different fine-grained layers on the performance of capillary barrier system. The capillary barriers consist of 20 cm thick of gravelly sand as the coarse-grained layer and 40 cm thick of fine-grained layer, which was residual soil (RS) or mixture of residual soil and fine sand (MRF_60-40) or fine sand (FS). The capillary barriers were subjected similar rainfall intensity of 15 mm/hr for 6 hours. To study the performance of the capillary barriers, especially the one using residual soil as the fine-grained layer, the pore-water pressure and volumetric water contents were analyzed.

4.9.2 Effect of evaporation on the capillary barrier using residual soil as the fine-grained layer

Effect of evaporation on the recovery of water storage of residual soil as the fine-grained layer was also investigated. A capillary barrier consisting of 40 cm thick of

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residual soil as the fine-grained layer and 20 cm thick of gravelly sand as the coarse-grained layer was subjected to different evaporation rates. The evaporation rates applied to the capillary barrier were 0 mm/d (RS40GS20-RF1-EV1), 3 mm/day (RS40GS20-RF1-EV2), and 6 mm/day (RS40GS20-RF1-EV3). The maximum evaporation rate simulated in this study (6 mm/d) was based on the potential evaporation in Singapore as calculated by Gasmø (1997), which was in the range of 5.16 to 7.53 mm/day. To study the effect of evaporation, the pore-water pressure and volumetric water contents were analyzed.

4.10 Concluding Remarks

The research programme of this study is explained in detail in this chapter. Methods for obtaining the unsaturated properties, such as water characteristic curves are described. The geotextile-water characteristic curve was obtained using the capillary rise principle. The equipments and measuring devices used in this study and the experimental and numerical programmes are outlined in detail in this chapter. The soil column and the capillary barrier models were subjected to infiltration and evaporation processes. The rainfall intensities applied to the models were based on the rainfall condition that caused slope failures in residual soil slopes in NTU (Rahardjo et al., 2001). The infiltration process was then followed with an evaporation process. The evaporation rates applied to the models were based on the potential evaporation rate for Singapore calculated by Gasmø (1997).

Chapter 5

PRESENTATION OF RESULTS

5.1 Introduction

Results of the experimental and numerical programmes described in Chapter 4 are presented in this chapter. The properties of the materials used in the study, such as basic soil-properties and soil-water characteristic curves, are presented in Section 5.2. In Section 5.3, results obtained from the evaporation and shrinkage tests are presented. Section 5.4 presents the experimental results of the soil column models. Experimental results of the capillary barrier models are presented in Section 5.5. Subsequently, numerical simulation results are presented in Section 5.6 and finally, results of parametric study are given in Section 5.7.

5.2 Material Properties

Properties of materials used in this study are presented in this section. Basic soil properties, such as grain-size distribution, specific gravity, Atterberg limits, compaction test results, and saturated coefficient of permeability are presented in Section 5.2.1. Water characteristic curves and permeability functions of the materials used in this study are presented in Section 5.2.2 and 5.2.3, respectively.

5.2.1 Basic soil properties

The residual soil (RS) used in this study was obtained from the Bukit Timah Granite formation which covers one-third of Singapore Island. The gravelly sand (GS) was a commercially available mixture of coarse sand (50.1%) and fine gravel (49.9%), while the fine sand (FS) was a sand used in Changi reclamation projects. Other two soils were obtained by mixing the residual soil and fine sand at different proportions by dry mass. The MRF₈₀₋₂₀ was obtained by mixing 80% residual soil and 20% fine sand, while MRF₆₀₋₄₀ was obtained by mixing 60% residual soil and 40% fine sand. Grain-size distribution of the soils used in this study are presented in Figure 5.1 and Figure 5.2.

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Figure 5.1 Grain-size distribution of the original soils

Figure 5.2 Grain-size distribution of the mixtures of residual soil and fine sand

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Other basic properties of the soils are listed in Table 5.1 and Table 5.2. The basic properties of the gravelly sand and fine sand were obtained previously in the studies conducted by Tami et al. (2004a) and Tami (2004)

Table 5.1 Basic properties of soils used in the study

NO	DESCRIPTION	UNIT	SOILS		
			GRAVELLY SAND (GS)	FINE SAND (FS)	RESIDUAL SOIL (RS)
1	Unified Soil Classification System		SP	SP	ML
2	Specific gravity, G_s		2.62	2.65	2.65
3	Grain-size analysis				
	- D_{60}	mm	5.15	0.35	0.015
	- D_{50}	mm	4.75	0.32	0.0068
	- D_{30}	mm	3.68	0.23	0.0025
	- D_{10}	mm	2.73	0.17	0.0007
	- Coefficient of uniformity, C_u		1.9	2.1	21.4
	- Coefficient of curvature, C_c		0.96	0.89	0.6
	- Gravel content (>4.75 mm)	%	49.9	0	0.7
	- Sand content (0.075 - 4.75 mm)	%	50.1	99.2	31.8
	- Fine content (<0.075 mm)	%	0	0.8	67.5
4	Atterberg limits				
	- Liquid limit, LL	%	-	-	47
	- Plastic limit, PL	%	-	-	32
	- Plasticity index, PI	%	-	-	15
5	Relative density				
	- Minimum dry density, $\rho_{d \text{ min}}$	Mg/m ³	1.42	1.34	-
	- Maximum dry density, $\rho_{d \text{ max}}$	Mg/m ³	1.71	1.61	-
	- Maximum void ratio, e_{max}		0.845	0.978	-
	- Minimum void ratio, e_{min}		0.532	0.646	-
6	Compaction test				
	- Maximum dry density, $\rho_{d \text{ max}}$	Mg/m ³	-	-	1.56
	- Optimum water content, w_{opt}	%	-	-	24
7	Saturated coefficient of permeability, k_s	m/s	7.6×10^{-2}	2.7×10^{-4}	1.55×10^{-7}
	at dry density, ρ_d	Mg/m ³	1.62	1.56	1.15

Table 5.2 Basic properties of soil mixtures used in the study

NO	DESCRIPTION	UNIT	SOILS		
			RS	MRF_80-20	MRF_60-40
1	Unified Soil Classification System		ML	ML	SM
2	Specific gravity, G_s		2.65	2.64	2.65
3	Grain-size analysis				
	- Gravel content (>4.75 mm)	%	0.65	0.25	0.08
	- Sand content (0.075 - 4.75 mm)	%	32.06	44.53	59.08
	- Fine content (<0.075 mm)	%	67.29	55.22	40.84
4	Atterberg limits				
	- Liquid limit, LL	%	47.0	41.4	33.8
	- Plastic limit, PL	%	32.0	27.0	22.1
	- Plasticity index, PI	%	15.0	14.4	11.7
5	Saturated coefficient of permeability, k_s	m/s	8.57E-08	1.59E-07	2.71E-07
	at dry density, ρ_d	Mg/m ³	1.15	1.16	1.15

Other than the Standard Proctor Compaction test, a compaction test using a vibratory compactor was also carried out to obtain the achieved dry density of the capillary barrier model. Figure 5.3 shows the results of the Standard Proctor Compaction test and the vibratory compaction test of the residual soil.

Figure 5.3 Standard Proctor Compaction and vibratory compaction curves of the residual soil

5.2.2 Water characteristic curves

The drying and wetting water characteristic curves of the materials used in this study are presented in this section. In Tempe cell test and Pressure plate tests, the volume of water during the drying process was plotted with respect to time to determine the equilibrium condition. Figure 5.4 shows a typical volume changes of water during a drying process in a Tempe cell test. The water content values at equilibrium condition under the applied matric suctions in the test were used to plot the water characteristic curves. These experimental data were fitted using Fredlund & Xing (1994) equation. As recommended by Leong and Rahardjo (1994), the correction factor of the fitting equation, $C(\psi)$, was set to 1.

Figure 5.4 Typical changes in volume of water during a drying process in Tempe cell and Pressure plate tests

The drying (D) and wetting (W) soil-water characteristic curves (SWCC) of the residual soil and mixture of residual soil and fine sand (MRF_60-40) used in the capillary barrier models are presented in Figure 5.5. The drying SWCC of MRF_80-20 is also presented in Figure 5.5. Figure 5.6 shows the drying and wetting SWCC of the fine sand and gravelly sand used in this study (Tami et al., 2004a). The best fitting parameters and properties of the SWCC of the soils are presented in Table

5.3. The properties of the gravelly sand (GS) and fine sand (FS) were obtained from previous study by Tami (2004).

Figure 5.5 Soil-water characteristic curves of the residual soil and soil mixtures

Figure 5.6 Soil-water characteristic curves of the fine sand and gravelly sand (Tami et al., 2004a)

Table 5.3 Parameters of the soil-water characteristic curves

DESCRIPTION	SYMBOL	SOILS			
		GS	FS	RS	MRF_60-40
Saturated volumetric water content	θ_s	0.382	0.411	0.555	0.525
Air-entry value (kPa)	ψ_a	0.110	1.400	2.748	0.468
Residual volumetric water content	θ_r	0.021	0.035	0.300	0.258
Residual matric suction (kPa)	ψ_r	0.401	3.940	163.598	40.209
Water-entry value (kPa)	ψ_w	0.285	2.688	42.949	5.568
Best-fitting parameters (Drying curve)	a	0.176	1.941	4.079	0.870
	n	4.440	6.302	2.231	1.446
	m	1.130	0.868	0.230	0.314
Best-fitting parameters (Wetting curve)	a	0.071	1.808	6.277	0.484
	n	1.232	3.193	1.204	1.935
	m	2.710	3.740	1.808	0.615

Other than the soils, water characteristic curves of the filter geotextile in the Polyfelt Megadrain 2040 were also determined. Capillary rise principle was used to obtain both drying and wetting geotextile-water characteristic curves (GWCC). The GWCC were obtained by plotting the water contents of each specimen from different heights against their corresponding matric suctions at equilibrium condition. As it was difficult to visually observe the equilibrium condition, the geosynthetic capillary rise tests were conducted by limiting the duration of the tests. The durations of the tests used in this study were 1, 2, 3, and 4 days. Figures 5.7 and 5.8 show the test results of drying and wetting GWCC under different durations, respectively.

The results showed that there are no significant differences from the tests under different durations. The results from one day duration were selected to be used for further analysis and numerical simulations. Figure 5.9 shows the drying and wetting water characteristic curves and the fitted curves of the filter geotextile. The best fitting parameters and properties of the GWCC are presented in Table 5.4.

Figure 5.7 Drying water characteristic curves of the filter geotextile

Figure 5.8 Wetting water characteristic curves of the filter geotextile

Figure 5.9 Drying and wetting water characteristic curves of the filter geotextile

Table 5.4 Parameters of the geotextile-water characteristic curves

DESCRIPTION	SYMBOL	FILTER GEOTEXTILE	
		DRYING	WETTING
Saturated volumetric water content	θ_s	0.843	
Air-entry value (kPa)	ψ_a	0.418	
Residual volumetric water content	θ_r	3.858E-05	
Residual matric suction (kPa)	ψ_r	1.936	
Water-entry value (kPa)	ψ_w	0.264	
Best-fitting parameters	a	1.504	0.213
	n	2.128	4.698
	m	5.918	4.542

5.2.3 Permeability functions

The permeability function of the soils used in this study was calculated using the statistical method presented in Fredlund and Rahardjo (1993a). Drying and wetting SWCCs' and laboratory determined saturated coefficient of permeability were used to generate the permeability functions. Figures 5.10 and 5.11 show the

permeability functions of the residual soil and the MRF_60-40 mixture, respectively. Permeability functions of the fine sand and gravelly sand (Figure 5.12) were obtained from a previous study by Tami et al. (2004a).

Figure 5.10 Drying and wetting permeability functions of the residual soil calculated using statistical method

Figure 5.11 Drying and wetting permeability functions of the MRF_60-40 calculated using statistical method

Figure 5.12 Drying and wetting permeability functions of the fine sand (FS) and gravelly sand (GS) (Tami et al., 2004a)

The permeability function of the soils used as fine-grained layers in the capillary barrier models were also calculated using the instantaneous profile method (Watson, 1966; Hillel et al., 1972 and Olson and Daniel, 1981; after Fredlund and Rahardjo, 1993a). Figures 5.13 and 5.14 show the results of the instantaneous profile method and the permeability functions generated based on the results of the instantaneous profile method for the residual soil and MRF_60-40 mixture, respectively.

5.3 Results of the Evaporation and Shrinkage Test

The evaporation and shrinkage characteristics of the residual soil and the soil mixtures were investigated by performing the evaporation and shrinkage tests as described in Chapter 4. The evaporation and shrinkage tests were conducted under three different drying conditions (i.e. an ambient condition, an accelerated evaporation using fan, and an accelerated evaporation using lamp).

Figure 5.13 Drying and wetting permeability functions of the residual soil calculated using statistical method based on the instantaneous profile results

Figure 5.14 Drying and wetting permeability functions of the MRF_60-40 calculated using statistical method based on the instantaneous profile results

Figures 5.15, 5.16, and 5.17 show the void ratio changes during the evaporation and shrinkage tests of the residual soil (RS), MRF_80-20, and MRF_60-40, respectively. Degree of saturation changes during the evaporation and shrinkage tests of the residual soil (RS), MRF_80-20, and MRF_60-40 are presented in Figures 5.18, 5.19, and 5.20, respectively.

Figure 5.15 Void ratio changes of the residual soil under three different drying conditions

Figure 5.16 Void ratio changes of the MRF_80-20 under three different drying conditions

Figure 5.17 Void ratio changes of the MRF_60-40 under three different drying conditions

Figure 5.18 Degree of saturation changes of the residual soil under three different drying conditions

Figure 5.19 Degree of saturation changes of the MRF_80-20 under three different drying conditions

Figure 5.20 Degree of saturation changes of the MRF_60-40 under three different drying conditions

It can be seen that generally the changes in void ratio and degree of saturation of the specimens under the evaporation using lamp is faster than in the specimens under the evaporation using fan. The rate of changes experienced by the specimens under the ambient conditions were the slowest.

Figures 5.21, 5.22, and 5.23 show the shrinkage curves of the residual soil, MRF_80-20, and MRF_60-40, respectively.

Figure 5.21 Shrinkage curve of the residual soil

Figure 5.22 Shrinkage curve of the MRF_80-20

Figure 5.23 Shrinkage curve of the MRF_60-40

It can be seen that even though the changes in void ratio and degree of saturation were different, the shrinkage curves were identical for the same soil under different drying conditions. The shrinkage test data were fitted using the equation proposed by Fredlund et al. (2002).

5.4 Results of Experimental Studies on Soil Column Models

This section presents test results on the soil column models. The soil column models were subjected to rapid drawdown tests followed by rainfall and drying tests. The results presented in this section are in the form of pore-water pressure head profiles.

5.4.1 Soil column model consisting of 0.45 m thick fine sand overlying 0.08 m thick gravelly sand (SCM_FS-GS)

Test results on SCM_FS-GS are presented in this section. Figure 5.24 shows the progression of pore-water pressure head (PWP head) profiles during rapid drawdown test. The rapid drawdown test was conducted by lowering the water table from the top of the soil column ($z = 1.0\text{m}$) to the bottom of the soil column (z

= 0). Initially, the column was at hydrostatic positive pore-water pressure condition (the soil column was fully saturated). Once the rapid drawdown test started, the PWP head started moving towards negative values. The PWP head profile stabilized after around 24 hours.

Figure 5.24 Pore-water pressure head profiles in Test SCM_FS-GS_DD (drawdown test)

Rainfall tests with different durations and intensities were applied to the soil column model. Figures 5.25, 5.26, and 5.27 show the PWP head profiles of SCM_FS-GS_RF1, SCM_FS-GS_RF2, and SCM_FS-GS_RF3 tests, respectively. It can be seen that the PWP head profile moving towards positive values during the wetting process and negative values during the drying process. PWP head of the gravelly sand and the fine sand below the gravelly sand were also affected during the wetting and drying processes.

5.4.2 Soil column model consisting of 0.51 m thick fine sand overlying a geosynthetic material (SCM_FS-GE)

Test results of SCM_FS-GE are presented in this section. Figure 5.28 shows the progression of PWP head profiles of SCM_FS-GE during the rapid drawdown test. The progression of PWP head in the rapid drawdown test of SCM_FS-GE was

slower than that of SCM_FS-GS. The PWP head profile of SCM_FS-GE_DD test stabilized after around 48 hours.

Figure 5.25 Pore-water pressure head profiles in Test SCM_FS-GS_RF1 (a) during rainfall of 5.8 mm/hr for 6 hr and (b) after rainfall

Figure 5.26 Pore-water pressure head profiles in Test SCM_FS-GS_RF2 (a) during rainfall of 10.7 mm/hr for 6 hr and (b) after rainfall

Figure 5.27 Pore-water pressure head profiles in Test SCM_FS-GS_RF3 (a) during rainfall of 4.9 mm/hr for 9 hr and (b) after rainfall

Figure 5.28 Pore-water pressure head profiles in Test SCM_FS-GE_DD (drawdown test)

Rainfall tests with different durations and intensities were also applied to SCM_FS-GE. Figures 5.29, 5.30, and 5.31 show the pore-water pressure head profiles of SCM_FS-GE_RF1, SCM_FS-GE_RF2, and SCM_FS-GE_RF3 tests, respectively.

Similar to the tests on SCM_FS-GS, PWP head profile of the fine sand above the geosynthetic on SCM_FS-GE also moved towards positive values during the wetting process and negative values during the drying process. However, the fine sand below the geosynthetic material was not affected during the wetting and drying processes, indicating that water did not infiltrate through the geosynthetic material.

Figure 5.29 Pore-water pressure head profiles in Test SCM_FS-GE_RF1 (a) during rainfall of 5.8 mm/hr for 6 hr and (b) after rainfall

Figure 5.30 Pore-water pressure head profiles in Test SCM_FS-GE_RF2 (a) during rainfall of 12.6 mm/hr for 6 hr and (b) after rainfall

Figure 5.31 Pore-water pressure head profiles in Test SCM_FS-GE_RF3 (a) during rainfall of 4.3 mm/hr for 9 hr and (b) after rainfall

5.5 Results of Experimental Studies on Capillary Barrier Models

This section presents test results on the capillary barrier models. The capillary barrier models were subjected to infiltrations and subsequently followed by evaporation processes under different evaporation conditions. The results presented in this section are in the form of pore-water pressure head (PWP head) and volumetric water content (VWC) changes and water balance of the capillary barrier models. Measurements of pore-water pressure and volumetric water content from three cross-sections (Figure 5.32) are presented.

Figure 5.32 Investigated cross sections of the capillary barrier models

5.5.1 Capillary barrier model consisting of 0.2 m thick residual soil overlying 0.2 m thick gravelly sand (CBM_RS-GS)

Results of infiltration and evaporation tests on CBM_RS-GS are presented in this section. The CBM_RS-GS was subjected to a rainfall of 9 mm/hr for 3 hours and then subsequently subjected to evaporation processes under different conditions (ambient condition, accelerated evaporation using fan, accelerated evaporation

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using lamp, and accelerated evaporation using lamp and fan). Figures 5.33, 5.35, 5.37, and 5.39 show the PWP head and VWC changes of tests CBM_RS-GS_RF1, CBM_RS-GS_RF2, CBM_RS-GS_RF3, and CBM_RS-GS_RF4, respectively. The water balances of the tests are presented on Figures 5.34, 5.36, 5.38, and 5.40. There are five components of water balance that were measured in this study, namely: precipitation, P_r ; run-off, R ; lateral diversion, L ; percolation, P_e ; and water storage changes, ΔS . The precipitation was the amount of rainfall applied to capillary barrier model, which was measured using the digital flowmeter. The run-off, lateral diversion, and percolation were obtained from the output drainage of the capillary barrier model, which was measured independently and automatically using three electronic weighing balances. The run-off was measured from the output drainage which came from the water flow on the surface of the capillary barrier model. The lateral diversion was measured from the output drainage generated by the fine-grained layer of the capillary barrier model, while the percolation was measured from the output drainage generated by the coarse-grained layer of the capillary barrier model. The water storage changes in the capillary barrier model were calculated from the volumetric water content data measured by the time domain reflectometry (TDR). The water storage changes equal to the changes in volumetric water content multiplied by the total volume of the fine-grained soil layer. The volumetric water content of the fine-grained layer was obtained by averaging the measurement from all TDRs in the layer.

From the water balances, it can be seen that generally there were no percolation and lateral diversion observed during the tests on CBM_RS-GS. The infiltrations were stored in the fine-grained layer with some runoff flow above the capillary barrier models.

Figure 5.33 Pore-water pressure head and volumetric water content changes in Test CBM_RS-GS_RF1 (rainfall of 8.95 mm/hr for 3 hr)

Figure 5.34 Water balance in Test CBM_RS-GS_RF1 (rainfall of 8.95 mm/hr for 3 hr)

Figure 5.35 Pore-water pressure head and volumetric water content changes in Test CBM_RS-GS_RF2 (rainfall of 9.01 mm/hr for 3 hr)

Figure 5.36 Water balance in Test CBM_RS-GS_RF2 (rainfall of 9.01 mm/hr for 3 hr)

Figure 5.37 Pore-water pressure head and volumetric water content changes in Test CBM_RS-GS_RF3 (rainfall of 8.88 mm/hr for 3 hr)

Figure 5.38 Water balance in Test CBM_RS-GS_RF3 (rainfall of 8.88 mm/hr for 3 hr)

Figure 5.39 Pore-water pressure head and volumetric water content changes in Test CBM_RS-GS_RF4 (rainfall of 9.04 mm/hr for 3 hr)

Figure 5.40 Water balance in Test CBM_RS-GS_RF4 (rainfall of 9.04 mm/hr for 3 hr)

5.5.2 Capillary barrier model consisting of 0.2 m thick mixture of residual soil and fine sand (MRF_60-40) overlying 0.2 m thick gravelly sand (CBM_MRF-GS)

Results of infiltration and evaporation tests on CBM_MRF-GS are presented in this section. Precipitations of around 10 mm/hr for 3 hours were applied in tests CBM_MRF-GS_RF1, CBM_MRF-GS_RF2, CBM_MRF-GS_RF3, and CBM_MRF-GS_RF4. The precipitations were then followed with evaporation under different drying conditions as listed in Table 4.5. Figures 5.41, 5.43, 5.45, and 5.47 show the PWP head and VWC changes of tests CBM_MRF-GS_RF1, CBM_MRF-GS_RF2, CBM_MRF-GS_RF3, and CBM_MRF-GS_RF4, respectively. The water balances of these tests are presented on Figures 5.42, 5.44, 5.46, and 5.48. The components of water balance measured in CBM_MRF-GS were similar to those measured in CBM_RS-GS, namely: precipitation, P_r ; run-off, R ; lateral diversion, L ; percolation, P_e ; and water storage changes, ΔS .

Six hours of precipitations were applied in tests CBM_MRF-GS_RF5, CBM_MRF-GS_RF6, and CBM_MRF-GS_RF7. Rainfall intensity of around 10 mm/hr was applied in tests CBM_MRF-GS_RF5 and CBM_MRF-GS_RF6, while an intensity of around 15 mm/hr was applied in tests CBM_MRF-GS_RF7. Figures 5.49, 5.51, and 5.53 show the PWP head and VWC changes of tests CBM_MRF-GS_RF5, CBM_MRF-GS_RF6, and CBM_MRF-GS_RF7, respectively. The water balances of these tests are presented on Figures 5.50, 5.52, and 5.54.

Figure 5.41 Pore-water pressure head and volumetric water content changes in Test CBM_MRF-GS_RF1 (rainfall of 10.26 mm/hr for 3 hr)

Figure 5.42 Water balance in Test CBM_MRF-GS_RF1 (rainfall of 10.26 mm/hr for 3 hr)

Figure 5.43 Pore-water pressure head and volumetric water content changes in Test CBM_MRF-GS_RF2 (rainfall of 9.68 mm/hr for 3 hr)

Figure 5.44 Water balance in Test CBM_MRF-GS_RF2 (rainfall of 9.68 mm/hr for 3 hr)

Figure 5.45 Pore-water pressure head and volumetric water content changes in Test CBM_MRF-GS_RF3 (rainfall of 10.09 mm/hr for 3 hr)

Figure 5.46 Water balance in Test CBM_MRF-GS_RF3 (rainfall of 10.09 mm/hr for 3 hr)

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Figure 5.47 Pore-water pressure head and volumetric water content changes in Test CBM_MRF-GS_RF4 (rainfall of 9.88 mm/hr for 3 hr)

Figure 5.48 Water balance in Test CBM_MRF-GS_RF4 (rainfall of 9.88 mm/hr for 3 hr)

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Figure 5.49 Pore-water pressure head and volumetric water content changes in Test CBM_MRF-GS_RF5 (rainfall of 9.85 mm/hr for 6 hr)

Figure 5.50 Water balance in Test CBM_MRF-GS_RF5 (rainfall of 9.85 mm/hr for 6 hr)

Figure 5.51 Pore-water pressure head and volumetric water content changes in Test CBM_MRF-GS_RF6 (rainfall of 9.88 mm/hr for 6 hr)

Figure 5.52 Water balance in Test CBM_MRF-GS_RF6 (rainfall of 9.88 mm/hr for 6 hr)

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Figure 5.53 Pore-water pressure head and volumetric water content changes in Test CBM_MRF-GS_RF7 (rainfall of 14.74 mm/hr for 6 hr)

Figure 5.54 Water balance in Test CBM_MRF-GS_RF7 (rainfall of 14.74 mm/hr for 6 hr)

5.5.3 Capillary barrier model consisting of 0.2 m thick fine sand overlying a geosynthetic material (CBM_FS-GE)

Results of rainfall tests comprising several stages and single event rainfalls on CBM_FS-GE are presented in this section. The rainfall intensity and durations are presented together with the PWP head and VWC changes on Figures 5.55, 5.57, 5.59, 5.61, 5.63, and 5.65. The A and D indicate adsorption and desorption processes, respectively. Pore-water pressure measurements at T-34 and T-44 were not available at some tests due to the faulty equipment.

Water balances of the tests are presented on Figures 5.56, 5.58, 5.60, 5.62, 5.64, and 5.66. In CBM_FS-GE, there are five components of water balance that were measured, namely: precipitation, Pr ; run-off, R ; lateral diversion, L ; geonet drainage, G ; and water storage changes, ΔS . Similar to the other capillary barrier model, the precipitation was the amount of rainfall applied to capillary barrier model, which was measured using the digital flowmeter. The run-off, lateral diversion, and geonet drainage were obtained from the output drainage of the capillary barrier model, which was measured independently and automatically using three electronic weighing balances. The run-off was measured from the output drainage which came from the water flow on the surface of the capillary barrier model. The lateral diversion was measured from the output drainage generated by the fine-grained layer (fine sand) of the capillary barrier model, while the geonet drainage was measured from the output drainage generated by the geonet drainage layer of the geosynthetic material. The water storage changes in the capillary barrier model were calculated from the volumetric water content data measured by the time domain reflectometry (TDR). The water storage changes equal to the changes in volumetric water content multiplied by the total volume of the fine-grained soil layer. The volumetric water content of the fine-grained layer was obtained by averaging the measurement from all TDRs in the layer.

In all tests of CBM_FS-GE, the run-off is zero because all the precipitation infiltrated into the fine sand due to its high coefficient of permeability.

Figure 5.55 Pore-water pressure head and volumetric water content changes in Test CBM_FS-GE_RF1

Figure 5.56 Water balance in Test CBM_FS-GE_RF1

Figure 5.57 Pore-water pressure head and volumetric water content changes in Test CBM_FS-GE_RF2 (rainfall of 10.07 mm/hr for 6 hr)

Figure 5.58 Water balance in Test CBM_FS-GE_RF2 (rainfall of 10.07 mm/hr for 6 hr)

Figure 5.59 Pore-water pressure head and volumetric water content changes in Test CBM_FS-GE_RF3

Figure 5.60 Water balance in Test CBM_FS-GE_RF3

Figure 5.61 Pore-water pressure head and volumetric water content changes in Test CBM_FS-GE_RF4

Figure 5.62 Water balance in Test CBM_FS-GE_RF4

Figure 5.63 Pore-water pressure head and volumetric water content changes in Test CBM_FS-GE_RF5 (rainfall of 9.83 mm/hr for 3 hr)

Figure 5.64 Water balance in Test CBM_FS-GE_RF5 (rainfall of 9.83 mm/hr for 3 hr)

Figure 5.65 Pore-water pressure head and volumetric water content changes in Test CBM_FS-GE_RF6 (rainfall of 31.07 mm/hr for 3 hr)

Figure 5.66 Water balance in Test CBM_FS-GE_RF6 (rainfall of 31.07 mm/hr for 3 hr)

5.6 Results of Numerical Simulations

Results of numerical simulation on the soil column models and capillary barrier models are presented in this section. Some test results are presented in this section and compared with the experimental results. Numerical simulations during rainfall were conducted using the wetting SWCC and the wetting permeability function, while numerical simulations after rainfall were conducted using the drying SWCC and the drying permeability function.

5.6.1 Numerical simulation of soil column models

This section presents the results of numerical simulation of soil column models (SCM_FS-GS and SCM_FS-GE). The results presented are in the form of pore-water pressure head profiles.

5.6.1.1 Numerical simulation of SCM_FS-GS

Figure 5.67 shows the comparison of numerical simulation and experimental results during the drawdown test (SCM_FS-GS_DD). Comparisons between numerical and experimental results of SCM_FS-GS_RF1 test during and after rainfall are presented on Figure 5.68 and 5.69, respectively.

The numerical model was able to simulate the drawdown and wetting processes closely to match the experimental data. However, some discrepancies are observed

on the fine sand above the gravelly sand during the drying process as shown in Figure 5.69.

Figure 5.67 Comparison of experimental and numerical results of pore-water pressure head profiles in Test SCM_FS-GS_DD (drawdown test)

Figure 5.68 Comparison of experimental and numerical results of pore-water pressure head profiles in Test SCM_FS-GS_RF1 during rainfall of 5.8 mm/hr for 6 hr

Figure 5.69 Comparison of experimental and numerical results of pore-water pressure head profiles in Test SCM_FS-GS_RF1 after rainfall of 5.8 mm/hr for 6 hr

5.6.1.2 Numerical simulation of SCM_FS-GE

Comparison of numerical simulation and experimental results of test SCM_FS-GE_DD is presented in Figure 5.70. Figures 5.71 and 5.72 show the comparison of numerical and experimental results in Test SCM_FS-GE_RF1 during and after rainfall, respectively.

It can be seen that the numerical simulation agree reasonably well with the experimental data except at the top part of the fine sand. During the drawdown and the drying process after rainfall, the PWP heads obtained from the numerical simulation results move faster towards the hydrostatic condition as compared to those obtained from the experiments.

Figure 5.70 Comparison of experimental and numerical results of pore-water pressure head profiles in Test SCM_FS-GE_DD (drawdown test)

Figure 5.71 Comparison of experimental and numerical results of pore-water pressure head profiles in Test SCM_FS-GE_RF1 during rainfall of 5.8 mm/hr for 6 hr

Figure 5.72 Comparison of experimental and numerical results of pore-water pressure head profiles in Test SCM_FS-GE_RF1 after rainfall of 5.8 mm/hr for 6 hr

5.6.2 Numerical simulation of capillary barrier models

This section presents the results of numerical simulation of capillary barrier models (CBM_RS-GS, CBM_MRF-GS and CBM_MRF-GS). The input parameters used in the numerical simulations is explained in detail in Section 4.8. The methods of determination for the parameters used in the numerical analyses (SWCC and permeability functions) are explained in Sections 4.3.2 and 4.3.3. The values of the input parameters are presented in Section 5.2. The numerical simulation results presented in this section are in the form of pore-water pressure head profiles.

5.6.2.1 Numerical simulation of CBM_RS-GS

Numerical simulation of CBM_RS_GS was conducted using SWCC (measured using tempe cell and capillary rise tests) and permeability functions (generated using SWCC and independently measured saturated coefficient of permeability, k_s) of the residual soil and gravelly sand. Figure 5.73 shows the comparison of experimental and numerical results of PWP head profiles at three cross-sections (see Figure 5.32) in Test CBM_RS-GS_RF1. It can be seen from 5.73 that the changes

of PWP head in the fine-grained layer from the numerical results were slower than the changes of PWP head from the experimental results. It was suspected that the saturated coefficient of permeability of the residual soil used in the capillary barrier model was higher than the saturated coefficient of permeability measured independently in the laboratory using the falling head test method. The instantaneous profile method was then adopted to obtain the permeability function of the residual soil used in the capillary barrier model. Results of the instantaneous profile method (Section 5.2.3) showed that the saturated coefficient of permeability of the residual soil in the capillary barrier model was one order of magnitude higher than the one measured using the falling head test. The permeability functions obtained from the instantaneous profile were then used in the numerical simulation.

Figure 5.73 Comparison of pore-water pressure head profiles during rainfall in Test CBM_RS-GS_RF1 (rainfall of 8.95 mm/hr for 3 hr) from experimental and numerical results with permeability function generated using independently measured k_s

Figure 5.74 shows the comparison of PWP head profiles from experimental data and numerical simulation results using the permeability functions obtained from the instantaneous profile method. It can be seen that the numerical results agree reasonably well with the experiment data. Therefore, subsequent numerical simulations were conducted using the permeability functions obtained from the instantaneous profile method.

Figure 5.74 Comparison of pore-water pressure head profiles during rainfall in Test CBM_RS-GS_RF1 (rainfall of 8.95 mm/hr for 3 hr) from experimental and numerical results with permeability function obtained from instantaneous profile method

The drying process in Test CBM_RS-GS_RF1 (ambient conditions) was also simulated. Figure 5.75 shows the comparison of PWP head profiles during the drying process. The numerical simulation was able to produce similar pore-water pressure head profiles at the end of the drying process under the laboratory ambient conditions.

Figure 5.75 Comparison of experimental and numerical results of pore-water pressure head profiles after rainfall in Test CBM_RS-GS_RF1 (rainfall of 8.95 mm/hr for 3 hr)

5.6.2.2 Numerical simulation of CBM_MRF-GS

Numerical simulation of CBM_MRF-GS was conducted using the SWCC (measured using tempe cell and capillary rise tests) and the permeability functions obtained from the instantaneous profile method (Figure 5.14). Figure 5.76 and 5.77 show the comparison of pore-water pressure head profiles during and after rainfall in Test CBM_MRF-GS_RF2, respectively. It can be seen that the numerical simulation was able to simulate the wetting process closely. However, the PWP heads at cross sections 1 and 3 from the experimental results were lower than the results from the numerical simulation during the drying process (under the accelerated evaporation using lamp and fan). It was suspected that the evaporation process using lamp and fan was not evenly distributed. Higher evaporation rates occurred at the edges of the capillary barrier model because of the placement of the fans at both ends of the model.

Figure 5.76 Comparison of experimental and numerical results of pore-water pressure head profiles during rainfall in Test CBM_MRF-GS_RF2 (rainfall of 9.68 mm/hr for 3 hr)

Figure 5.77 Comparison of experimental and numerical results of pore-water pressure head profiles after rainfall in Test CBM_MRF-GS_RF2 (rainfall of 9.68 mm/hr for 3 hr)

5.6.2.3 Numerical simulation of CBM_FS-GE

Comparison of pore-water pressure head profiles from experimental and numerical results during and after rainfall in Test CBM_FS-GE_RF2 are presented in Figure 5.78 and 5.79, respectively. The wetting process could be simulated reasonably well. However, the PWP head from the numerical simulation results during the drying process was lower than those from the experimental data.

Figure 5.78 Comparison of experimental and numerical results of pore-water pressure head profiles during rainfall in Test CBM_FS-GE_RF2 (rainfall of 10.07 mm/hr for 6 hr)

Figure 5.79 Comparison of experimental and numerical results of pore-water pressure head profiles after rainfall in Test CBM_FS-GE_RF2 (rainfall of 10.07 mm/hr for 6 hr)

5.7 Results of Parametric Studies

Results of parametric studies are presented in this section. The parametric studies were conducted mainly to investigate the behaviour of capillary barriers constructed using residual soil as the fine-grained layer.

5.7.1 Comparison of capillary barriers using different fine-grained layers

Effect of different fine-grained layers on the behaviour of capillary barrier system were investigated. Three numerical simulations; namely RS40GS20-RF1-EV1, MRF40GS20-RF1-EV1, and FS40GS20-RF1-EV1 were conducted for this purpose. The capillary barriers consist of 20 cm thick of gravelly sand as the coarse-grained layer and 40 cm thick of fine-grained layer, which was residual soil (RS) or mixture of residual soil and fine sand (MRF) or fine sand (FS). The capillary barriers were subjected to a rainfall intensity of 15mm/hr for 6 hours.

Figures 5.80 and 5.81 present the PWP head and volumetric water content profiles of the capillary barriers at three different cross sections. The profiles are presented at $t=0$ (initial conditions) and at $t=6$ (end of rainfall).

Figure 5.80 Comparison of pore-water pressure head profiles for capillary barrier models with different fine-grained layers

Figure 5.81 Comparison of volumetric water content profiles for capillary barrier models with different fine-grained layers

5.7.2 Effect of evaporation on the capillary barrier using residual soil as the fine-grained layer

Effect of evaporation on the recovery of residual soil as the fine-grained layer was also investigated in this parametric study. A capillary barrier of a 40 cm thick of residual soil overlying a 20 cm thick gravelly sand was subjected to different evaporation rates. The evaporations rates applied to the capillary barrier were 0 mm/day (RS40GS20-RF1-EV1), 3 mm/day (RS40GS20-RF1-EV2), and 6 mm/day (RS40GS20-RF1-EV3). The maximum evaporation rate simulated in this study (6mm/day) was based on the potential evaporation in Singapore as calculated by Gasmu (1997), which was in the range of 5.16 to 7.53 mm/day.

Figures 5.82 and 5.83 present the PWP head and volumetric water content profiles of the capillary barrier at three different cross sections. The profiles are presented at t=0 (end of rainfall), which are the same for all simulations, t=1 day, and t=5 days.

Figure 5.82 Comparison of pore-water pressure head profiles for capillary barrier models under different evaporation rate

Figure 5.83 Comparison of volumetric water content profiles for capillary barrier models under different evaporation rate

5.8 Concluding Remarks

The experimental and numerical analysis results are presented in this chapter. The reasonably good results obtained from the experiments on the soil column and the capillary barrier models indicate that the equipments and measuring devices used in this study worked well. The numerical analyses adopted in this study could simulate the laboratory experiment closely. Numerical analyses of the capillary barrier models using saturated coefficient of permeability obtained from the instantaneous profile method show good agreement with the experimental data.

Chapter 6

DISCUSSIONS

6.1 Introduction

In this chapter, the results presented in Chapter 5 are discussed. The results of evaporation and shrinkage tests on the residual soil and the mixtures of residual soil and fine sand are discussed in Section 6.2. Section 6.3 discusses the residual soil as the fine-grained layer in capillary barrier. The performance of mixture of residual soil and fine sand (MRF_60-40) as the fine-grained layer in capillary barrier is discussed in Section 6.4. Finally, the mechanism and performance of capillary barriers using a geosynthetic material as the coarse-grained layer is discussed in Section 6.5.

6.2 Evaporation and Shrinkage Characteristics of Residual Soil and Mixtures of Residual Soil and Fine Sand

Evaporation and shrinkage tests were conducted to obtain the evaporation and shrinkage characteristics of the residual soil (RS) and mixtures of residual soil and fine sand (MRF_80-20 and MRF_60-40). The evaporation and shrinkage characteristics were used to assess the possibility of using the soils as fine-grained layer of capillary barriers. Soil specimens of 50 mm thick and 103 mm diameter were used in the evaporation and shrinkage tests. The specimens were subjected to three different evaporation conditions, which were ambient conditions, an accelerated evaporation using fan, and an accelerated evaporation using lamp.

6.2.1 Effect of evaporation rate

The changes in degree of saturation of the residual soil (RS), MRF_80-20, and MRF_60-40 mixtures under three different drying conditions are presented in Figures 5.18, 5.19, and 5.20, respectively. As expected, specimens under the accelerated evaporation using a lamp desaturated faster than specimens under the accelerated evaporation using a fan. Subsequently, specimens under the

accelerated evaporation using a fan also desaturated faster than specimens under the ambient conditions. A similar trend was also observed for the changes in void ratio as presented in Figures 5.15, 5.16, and 5.17. This behaviour is due to the fact that the evaporation rates of the tests under the accelerated evaporation using lamp were the highest as compared to the evaporation rates under other drying conditions as can be seen in Table 6.1.

Table 6.1 Average potential and actual evaporations measured in the evaporation and shrinkage tests under different drying conditions

SOILS	AMBIENT		FAN		LAMP	
	PE	AE	PE	AE	PE	AE
RS	0.050	0.035	0.099	0.052	0.163	0.124
MRF_80-20	0.042	0.024	0.121	0.061	0.141	0.118
MRF_60-40	0.046	0.036	0.123	0.090	0.164	0.122
Note: PE = Potential evaporation (mm/hr) AE = Actual evaporation (mm/hr)						

Table 6.1 indicated that for all soil types, the actual evaporations increase as the potential evaporations increase. A relationship between the actual and potential evaporations in the evaporation and shrinkage tests could be developed based on Table 6.1. Figure 6.1 shows the plot of the measured average AE versus the measured average PE for all the evaporation and shrinkage tests. On average, the amount of actual evaporation was 71% of the potential evaporation. This value is in agreement with the magnitude obtained from a previous study by Rosenberg et al. (1983) (after Wilson, 1990). Rosenberg et al. (1983) reported that the rate of actual evapotranspiration (AE) is typically 60% to 90% of the rate from pan measurement (PE).

Figure 6.1 Relationship between the average AE and PE during the evaporation and shrinkage tests

Figures 5.21, 5.22, and 5.23 show the shrinkage curves of the soils investigated in this study. The shrinkage curves are presented in a plot of e (void ratio) versus w (gravimetric water content). Data from three different drying conditions (different evaporation rates) for the same soil are presented in one graph. It can be seen that data from different drying conditions produced a unique shrinkage curve for each soil. The shrinkage test data follow a similar path from the initial void ratio to the final void ratio. This result indicates that the shrinkage curve of a soil is not affected by the evaporation rates. The change in void ratio of a soil is mainly controlled by the change in water content of the soil.

Fredlund and Rahardjo (1993) and Fredlund et al. (2002) suggested that the shrinkage curve can be combined with the SWCC to obtain a relationship between void ratio and matric suction. The water content data from the shrinkage curves (Figures 5.21, 5.22, and 5.23) were used to obtain their corresponding matric suctions on the drying SWCC (Figure 5.5). Figures 6.2, 6.3, and 6.4 present the relationship between void ratio and matric suction for the residual soil, MRF_80-20, and MRF_60-40 mixtures, respectively. It can be seen that the drying conditions/ evaporation rates do not affect the relationship between void ratio and

matric suction. A unique relationship between void ratio and matric suction was obtained for each soil. It can be concluded that the void ratio of a soil is a unique function of water content or matric suction in the soil.

Figure 6.2 Volume change relationship of the residual soil

Figure 6.3 Volume change relationship of MRF_80-20

Figure 6.4 Volume change relationship of MRF_60-40

6.2.2 Effect of mixing the residual soil with fine sand

The residual soil was mixed with fine sand at different percentages in order to improve the hydraulic properties and reduce the shrinkage potential of the soil. It was expected to obtain a soil with good drainage capability and low shrinkage behaviour.

Figure 6.5 shows the fitted curves of the shrinkage curves (Figures 5.21, 5.22, and 5.23) of the soils investigated in this study. Based on the fitted curves, the volumetric strain of the soils ($\Delta e/(1 + e_0)$) were calculated and summarized in Table 6.2. The shrinkage behaviour of the residual soil appears to improve as fine sand is added to the residual soil. The volumetric strain of the MRF_60-40 is the lowest as compared to those of MRF_80-20 and the original residual soil. This is an indication that the MRF_60-40 shrinks less as compared to MRF_80-20 and the original residual soil.

Figure 6.5 Shrinkage curves for three different soil

Table 6.2 Volumetric strain of the soils

SOILS	e_0	e_f	$\Delta e/(1+e_0)$
RS	1.40	0.86	0.23
MRF_80-20	1.28	0.82	0.20
MRF_60-40	1.12	0.73	0.18

Figure 6.6 shows the comparison of the degree of saturation changes of three soils under the accelerated evaporation using fan. It can be seen that MRF_60-40 desaturated faster than the MRF_80-20 and subsequently followed by the residual soil. This is an indication that the MRF_60-40 releases water faster than the MRF_80-20 and the residual soil. This behaviour could be attributed to the increase in the saturated coefficient of permeability of the soil as fine sand is added to the residual soil (Table 5.2).

Figure 6.6 Degree of saturation changes of three different soil under accelerated evaporation using fan

The effect of mixing the residual soil and fine sand is also studied based on the ratio of AE and PE. Figure 6.7 presents the relationships between AE and PE for each soil during the evaporation and shrinkage tests. The average AE/PE ratio of the RS and MRF_80-20 were almost the same (69%), while the average AE/PE ratio of MRF_60-40 was 74%. This is an indication that the properties of the residual soil was not significantly improved by mixing it with fine sand at a proportion of 80% residual soil and 20% fine sand (MRF_80-20).

6.2.3 Concluding remarks

This evaporation and shrinkage test results indicated that shrinkage curve of a soil is not affected by the evaporation rates. Void ratio of a soil is a unique function of water content or matric suction in the soil.

The test results suggested that mixing the residual soil with fine sand improved the shrinkage characteristics and hydraulic properties of the soil. However, as indicated by the basic properties of the soils (Table 5.2), shrinkage curves (Figure 6.5), degree of saturation changes (Figure 6.6), and the ratio of AE/PE (Figure 6.7), the properties of the residual soil was not significantly improved by mixing it with

fine sand at a proportion of 80% residual soil and 20% fine sand (MRF_80-20). The MRF_60-40 shows more significant improvement in terms of the shrinkage characteristics and hydraulic properties. Therefore, MRF_60-40 was selected to be used as the fine-grained layer in one of the capillary barrier models.

Figure 6.7 Relationships between AE and PE for each soil during the evaporation and shrinkage tests

6.3 Residual Soil as a Fine-grained Layer in Capillary Barrier

One of the objectives of this research is to investigate the application of residual soil as fine-grained layer in capillary barrier. A capillary barrier model was constructed using the residual soil obtained from Bukit Timah Granite formation as the fine-grained layer and gravelly sand as the coarse-grained layer (CBM_RS-GS). Several infiltration and evaporation tests were applied to the capillary barrier models. Performance of the capillary barrier models during infiltration process is discussed in Section 6.3.1.

Previous research works by Tami (2004) indicated that the recovery of water storage in a capillary barrier constructed using a residual soil as the fine-grained layer was very slow. The infiltrating water was retained in the residual soil as there was no lateral diversion observed in the capillary barrier model due to the low

coefficient of permeability of the residual soil. Another means of water removal from the fine-grained layer is by evaporation. The effect of evaporation on the recovery of water storage from the residual soil layer is discussed in Section 6.3.2.

6.3.1 Performance of CBM_RS-GS during infiltration process

Performance of CBM_RS-GS during infiltration process was examined based on the measurement of pore-water pressures, volumetric water contents, water balances, and the water storage capacity. Figure 6.8 presents the pore-water pressure head profiles during rainfalls of Tests CBM_RS-GS_RF1. It can be seen that the pore-water pressure head of the fine-grained layers increased continuously while the pore-water pressure head of the coarse-grained layers remained constant. Measurement of volumetric water content by the TDR in the same test is presented in Figure 6.9. The measurement indicated that the volumetric water content in the fine-grained layer increased significantly during rainfall while the changes in the volumetric water content of the coarse-grained layer were very small. The changes were around 2% which is the accuracy of the TDR system (SoilMoisture Equipment Corp., 1996).

The measurements of pore-water pressure and volumetric water content indicated that the infiltrating water was held in the fine-grained layer. The water balance of the capillary barrier model (Figure 5.34) also supported this conclusion. During the tests, there was no water flowing out from the coarse-grained layer. Similar trends were also observed in other tests of CBM_RS-GS. There were no percolations observed in the tests as indicated in the water balances (Figures 5.36, 5.38, and 5.40).

Figure 6.8 Pore-water pressure head profiles during rainfall in Test CBM_RS-GS_RF1

Figure 6.9 Volumetric water content changes in Test CBM_RS-GS_RF1

Another criterion to determine the performance of capillary barrier is the water storage capacity of the fine-grained layer before percolation into the coarse-grained

layer occurs. The maximum storage of the fine-grained layer is equal to the volume of pores in the fine-grained layer (porosity $\times$ volume of the fine-grained layer = $0.441 \times 200 \text{ L} = 88.2 \text{ L}$ which is equivalent to 88.2 mm height of water). However, generally percolation into the coarse-grained layer occurs before the maximum storage of the fine-grained layer is reached.

A method to calculate the water storage capacity of capillary barriers was proposed by Stormont and Morris (1998). The method includes the following steps:

1. Estimate the matric suction profile in capillary barrier at breakthrough/percolation. The matric suction profile is assumed to be hydrostatic, starting from water-entry value of the coarse-grained layer.
2. Determine the volumetric water content profile based on the matric suction profile and the wetting soil-water characteristic curve of the soil used in the fine-grained layer.
3. Integrating the volumetric water content over the depth to obtain the water storage capacity of the capillary barrier.

This method was used to calculate the water storage capacity of CBM_RS-GS. Figure 6.10 illustrates the calculation of water storage capacity of CBM_RS-GS. Based on this method, the water storage capacity of CBM_RS-GS is 79 L or equal to 90 % of the maximum storage of 88.2 L.

The amount of water stored in the capillary barrier model was calculated using the volumetric water content measured by the TDR. The stored water is equal to the water content multiplied by the total volume of the residual soil layer. The volumetric water content of the residual soil layer was obtained by averaging the measurement from all TDRs in the layer. Based on the calculation, the amount of water stored in the residual soil layer during the tests in CBM_RS-GS was between 70 to 76 L. The highest amount of water stored in the residual soil layer was measured during Test CBM_RS-GS_RF2. Figure 6.11 and 6.12 show the volumetric water content measured by the TDRs and the amount of water stored in the residual soil layer during Test CBM_RS-GS_RF2, respectively.

Figure 6.10 Matric suction and volumetric water content profile to estimate the storage capacity of CBM_RS-GS

Figure 6.11 Volumetric water content changes in Test CBM_RS-GS_RF2

Figure 6.12 Total water storage in Test CBM_RS-GS_RF2

The amount of water stored during the tests in CBM_RS-GS (70 to 76 L) was lower than the water storage capacity of CBM_RS-GS calculated using the method proposed by Stormont and Morris (1998) (79.3 L). This is the reason that no significant percolation observed during the tests in CBM_RS-GS. The accuracy of calculation method proposed by Stormont and Morris (1998) is discussed in Section 6.5.2.

6.3.2 The effect of evaporative flux to CBM_RS-GS

Four tests with different evaporation conditions were applied to CBM_RS-GS in order to study the effect of evaporative flux on the recovery of the water storage in the residual soil. The tests were initiated with a rainfall of 9 mm/hr for 3 hours. The rainfall was then followed by evaporation under the following conditions: ambient conditions (CBM_RS-GS_RF1), an accelerated evaporation using lamp (CBM_RS-GS_RF2), an accelerated evaporation using fan (CBM_RS-GS_RF3), and an accelerated evaporation using both lamp and fan (CBM_RS-GS_RF4)).

As there was no lateral diversion flow observed in the water balance, the water content changes in the residual soil layer were caused by evaporation. The amount

of evaporation (E_T) from the residual soil layer in the capillary barrier model was calculated using the volumetric water content data measured by TDR. The amount of evaporation is formulated as follows:

$$E_T = \frac{\Delta\theta_w \cdot V_T}{A_T} \quad (6.1)$$

Where:

E_T = Amount of evaporation (mm)

$\Delta\theta_w$ = volumetric water content changes

V_T = volume of the fine-grained soil layer (mm³)

A_T = surface area of the evaporated soil layer (mm²)

The volumetric water content was obtained by averaging the measurements from all TDRs in the residual soil layer.

Potential evaporations during the tests were also measured. Potential evaporation is defined as the amount of water evaporated from a free water surface. Potential evaporation was measured simultaneously during the tests by measuring the changes in the mass of water in plastic beakers under the same drying conditions. Three plastic beakers were placed at the bottom, middle, and top of the capillary barrier model for the potential evaporation measurement.

The average rates of potential evaporation (PE) and actual evaporation (AE) during the tests are summarized in Table 6.3. From the table, it can be seen that the laboratory ambient conditions resulted in very low potential and actual evaporations as compared to the other modes of evaporations. The highest potential and actual evaporations were obtained in Test CBM_RS-GS_RF4 (accelerated evaporation using lamp and fan).

A relationship between the actual and potential evaporations could be developed based on Table 6.3. Figure 6.13 shows the plot of the measured average AE versus the measured average PE for four different evaporation conditions applied to the capillary barrier model. Based on Figure 6.13, the average AE/PE ratio obtained from the CBM_RS-GS is 73%. This value is reasonably close to the AE/PE ratio of

the residual soil in the evaporation and shrinkage tests as presented in Figure 6.7 (AE/PE = 69%).

Table 6.3 Average rates of the potential and actual evaporations during the tests in CBM_RS-GS

TEST	MODE OF EVAPORATION	PE (mm/hr)	AE (mm/hr)
CBM_RS-GS_RF1	Ambient conditions	0.023	0.017
CBM_RS-GS_RF2	Lamp	0.203	0.142
CBM_RS-GS_RF3	Fan	0.120	0.092
CBM_RS-GS_RF4	Lamp and fan	0.217	0.163

Figure 6.13 Relationship between AE and PE in CBM_RS-GS

Figure 6.14 presents the water storage changes during the tests of CBM_RS-GS. Water storage changes in the capillary barrier model is defined as the changes in the amount of water that is being stored in the fine-grained layer of the capillary barrier model. From Figure 6.14, it can be seen that under laboratory ambient conditions (CBM_RS-GS_RF1), the amount of water stored in the fine-grained layer did not change much during the drying process. The applied evaporative fluxes in Tests CBM_RS-GS_RF2, CBM_RS-GS_RF3, and CBM_RS-GS_RF4 did increase the

rate of water storage recovery in the residual soil. The average rates of water storage recovery during the tests in CBM_RS-GS are equal to the rates of actual evaporation as presented in Table 6.3. As the surface area of the capillary barrier model is about 1m², an actual evaporation rate of 0.017 mm/hr in Test CBM_RS-GS_RF1 is equal to the rate of changes in water storage of 0.017 L/hr.

Figure 6.14 Water storage changes during the tests in CBM_RS-GS

Based on Table 6.3, the highest rate of water storage changes in CBM_RS-GS was 0.163 L/hr (in Test CBM_RS-GS_RF4). This value is much lower than to the rate of water storage recovery in fine sand when used as the fine-grained layer. Based on the water balance from a test with a comparable rainfall intensity (Test CBM_FS-GE_RF5; Figure 5.64), the average rate of water storage recovery in fine sand was 4 L/hr during the first 5 hours of drying process and then the rate decreased because the water content in the fine sand became very low.

Other than increasing the rate of water storage recovery in the residual soil, the applied evaporative fluxes to CBM_RS-GS also resulted in excessive desiccation cracks. Figure 6.15 shows the desiccation cracks occurred after the drying process in Test CBM_RS-GS_RF4. These cracks will allow water to infiltrate into the residual soil layer easily and then percolate into the gravelly sand layer.

Figure 6.15 Cracks observed after the drying process in Test CBM_RS-GS_RF4

6.3.3 Numerical analyses and parametric studies on the capillary barrier using residual soil

Numerical analyses were conducted to simulate the tests on CBM_RS-GS. Independently measured SWCC (measured using tempe cell and capillary rise tests) and permeability function obtained using the instantaneous profile method were used for the numerical analyses (refer to Section 5.6.2.1). The pore-water pressure head profiles of Test CBM_RS-GS_RF1 obtained from the experimental and numerical results during and after rainfall are presented in Figures 5.74 and 5.75, respectively. It can be seen from Figures 5.74 and 5.75 that the numerical results agree reasonably well with the experimental data.

Using the verified numerical model, several tests were conducted to study the characteristics of residual soil as the fine-grained layer in capillary barrier system. Three capillary barriers comprising the residual soil, fine sand, and a mixture of residual soil and fine sand (MRF_60-40) as the fine grained layers and gravelly sand as the coarse-grained layer were analyzed. An identical initial pore-water pressure head was applied to the capillary barrier models. Similar rainfall intensities of 15 mm/hr for 6 hours were applied to the capillary barriers. The pore-water pressure head and volumetric water content profiles of the capillary

barriers at three different cross sections are presented in Figure 5.80 and 5.81, respectively. It can be seen from Figure 5.80 that capillary break was observed in the capillary barrier using residual soil as the fine-grained layer. It was able to maintain highly negative pore-water pressure head at the interface of fine-grained and coarse-grained layer. The pore-water pressure heads at the interface of fine-grained and coarse-grained layer of capillary barriers using fine sand and the MRF_60-40 increased significantly to almost zero, which may lead to percolation when subsequent rainfalls occur.

Recovery of water storage of the residual soil during drying process was also investigated in this parametric study. A capillary barrier comprising a 40 cm thick of residual soil overlying a 20 cm thick gravelly sand was subjected to three different evaporation rates. The evaporation rates applied to the capillary barrier were 0 mm/day, 3 mm/day, and 6 mm/day. The maximum evaporation rate simulated in this study (6mm/day) was in the range of the potential evaporation in Singapore as calculated by Gasmo (1997) (5.16 to 7.53 mm/day). The pore-water pressure head and volumetric water content profiles of the capillary barrier at three different cross sections are presented in Figures 5.82 and 5.83, respectively. The pore-water pressure head and volumetric water content profiles are presented at $t=0$ (end of the preceding rainfall), $t=1$ day, and $t=5$ days. It can be seen from Figure 5.82 that the pore-water pressure head of the residual soil only decreased slightly at the elapsed time of 1 day. A more significant decrease in the pore-water pressures was observed after 5 days of evaporation. However, even though the pore-water pressure head decreased quite significantly after 5 days, the volumetric water content did not change much as indicated in Figure 5.83. Similar to the observation from the experimental results of CBM_RS-GS, the results of parametric study also indicated that the recovery of water storage in the residual soil is very slow which is not favorable for a fine-grained layer in capillary barrier system.

6.3.4 Concluding remarks

The results of experimental and parametric study on the capillary barrier using residual soil as the fine-grained layer showed that the capillary barrier could

function well during infiltration process by storing the infiltrating water in the fine-grained layer up to its water storage capacity without significant percolation. However, the recovery rate of water storage in the residual soil was very slow. Water was held in the residual soil and did not flow laterally along the fine-grained layer because of the low coefficient of permeability of the residual soil. The infiltrating water was only removed by evaporation during the drying process. In addition, the laboratory capillary barrier model showed excessive desiccation cracks in the residual soil during drying process. The cracks could allow water to infiltrate into the residual soil layer easily and then percolate into the gravelly sand layer. This situation has to be avoided in a capillary barrier system.

The slow water storage recovery and excessive desiccation cracks will affect the long term performance of the capillary barrier. Therefore, the use of residual soil as the fine-grained layer of a capillary barrier system is not recommended.

6.4 Mixture of Residual Soil and Fine Sand (MRF₆₀₋₄₀) as a Fine-grained Layer in Capillary Barrier

The second capillary barrier model was constructed using a mixture of residual soil and fine sand (MRF₆₀₋₄₀) as the fine-grained layer and gravelly sand as the coarse-grained layer (CBM_{MRF-GS}). Several infiltration and evaporation tests were applied to the capillary barrier models (Table 4.4). Performance of CBM_{MRF-GS} during infiltration process is discussed in Section 6.4.1 and the recovery of water storage in CBM_{MRF-GS} is discussed in Section 6.4.2.

6.4.1 Performance of CBM_{MRF-GS} during infiltration process

Performance of CBM_{MRF-GS} during infiltration process was also examined based on the measurement of pore-water pressures, volumetric water contents, water balances, and the water storage capacity. Figure 6.16 presents the pore-water pressure head profiles during rainfalls of Tests CBM_{RS-GS}_RF2. It can be seen that the pore-water pressure heads in the fine-grained layers increased continuously while the pore-water pressure heads in the coarse-grained layers remained constant. Measurement of volumetric water content by the TDR in the fine-grained layer (MRF₆₀₋₄₀) and the coarse-grained layer (gravelly sand) are

presented in Figures 6.17 and 6.18, respectively. The measurements indicated that the volumetric water content in the fine-grained layer increased significantly during rainfall while the changes in the volumetric water content of the coarse-grained layer were very small. The maximum changes were around 2% which is within the measurement accuracy of the TDR system (SoilMoisture Equipment Corp., 1996). The measurements of pore-water pressure and volumetric water content indicated that the infiltrating water was retained in the fine-grained layer.

The performance of capillary barrier was also evaluated based on the water storage capacity of the fine-grained layer before percolation into the coarse-grained layer occurred. As discussed in Section 6.3.1, Stormont and Morris and (1998) proposed a method to calculate the water storage capacity of capillary barriers. This method was also used to calculate the water storage capacity of CBM_MRF-GS. Figure 6.19 illustrates the calculation of water storage capacity of CBM_MRF-GS. Based on this method, the water storage capacity of CBM_MRF-GS is 53 L.

Figure 6.16 Pore-water pressure head profiles during wetting process in Test CBM_MRF-GS_RF2

Figure 6.17 Volumetric water content changes in the MRF_60-40 layer during Test CBM_MRF-GS_RF2

Figure 6.18 Volumetric water content changes in the gravelly sand layer during Test CBM_MRF-GS_RF2

Figure 6.19 Matric suction and volumetric water content profile to estimate the storage capacity of CBM_MRF-GS

Further tests on CBM_MRF-GS indicated that percolations into the gravelly sand layer had occurred. The percolations were indicated by a significant increase in the volumetric water contents measured by TDRs in the gravelly sand layer. Figure 6.20 illustrates the increase in the volumetric water contents measured by the TDRs in the gravelly sand layer. The percolation into the gravelly sand layer in CBM_MRF-GS occurred due to the low water storage capacity of MRF_60-40. The water storage capacity of MRF_60-40 (53 L) is much lower than the water storage capacity of the residual soil in CBM_RS-GS (79 L). In addition, as indicated by the water balances, the amount of water that infiltrated into the MRF_60-40 was higher than the amount of water that infiltrated into the residual soil. For example, in the tests with a similar rainfall intensity, the percentage of water that infiltrated into the MRF_60-40 was 77% of the applied precipitation (in Test CBM_MRF-GS_RF2), while the percentage of water that infiltrated into the residual soil was 67% of the applied precipitation (in Test CBM_RS-GS_RF4). This behaviour was due to the fact that the coefficient of permeability of MRF_60-40 was higher than that of the residual soil.

Figure 6.20 Volumetric water content changes in the gravelly sand layer during Test CBM_MRF-GS_RF7

6.4.2 Recovery of water storage in CBM_MRF-GS

Figure 6.21 shows the water storage changes during the tests on CBM_MRF-GS. Results of three tests (CBM_MRF-GS_RF2, CBM_MRF-GS_RF3, and CBM_MRF-GS_RF4) are presented due to the similarity of rainfall intensity and duration applied to the capillary barrier models (refer to Table 4.4). It can be seen from Figure 6.21 that the water storage changes in CBM_MRF-GS behave differently as compared to the water storage changes in CBM_RS-GS (Figure 6.14). Sharp decreases of water storage were observed in CBM_MRF-GS, immediately after rainfall stopped. The water storage changes in CBM_MRF-GS behave similarly as the water storage changes in the fine sand layer as observed in the tests of CBM_FS-GE. A sharp decrease of water storage at the beginning of drying process is associated with the lateral diversion flow in the fine-grained layer.

Figure 6.21 Water storage changes during the tests in CBM_MRF-GS

The occurrence of lateral diversion flow in CBM_MRF-GS is also supported by the volumetric water content measured by the TDRs. The volumetric water content changes in the fine grained layer during Test CBM_MRF-GS_RF2 are presented in Figure 6.17. It can be seen that the initial volumetric water content in the lowermost part of the model (TDR_12) was the highest ($\theta_w = 0.29$) and the initial volumetric water content in the uppermost part of the model (TDR_72) was the lowest ($\theta_w = 0.21$). This is an indication that there was a lateral diversion flow from the upper part to the lower part of the capillary barrier model. The measurements of volumetric water content during and after rainfall at Test CBM_MRF-GS_RF2 also indicated the occurrence of lateral diversion flow in CBM_MRF-GS. During rainfall, the volumetric water content measured in the lower part of CBM_MRF-GS (TDR-12) increased to a higher value than those measured in the upper part (TDR-42; TDR-72). After rainfall, the volumetric water content in the lowest part of the model (TDR-12) remained constant while the volumetric water content measured in the upper part (TDR-42) decreased immediately after rainfall stopped. These measurements indicated that lateral diversion flow occurred during the entire duration of Test CBM_MRF-GS_RF2.

Even though there was a lateral diversion flow as indicated by the water storage changes and the volumetric water content measurements, lateral diversion flow was not observed in the lateral diversion measurement using weighing balance. It is suspected that water was held in the lowest part of the capillary barrier model as indicated by the constantly high volumetric water content as measured by TDR-12 (Figure 6.17). The infiltrating water in MRF_60-40 layer only flowed in a short distance because the coefficient of permeability of MRF_60-40 was still very low ($k_s = 5 \times 10^{-6}$ m/s; see Figure 5.14).

6.4.3 Shrinkage behaviour of MRF_60-40 in CBM_MRF-GS

During the experiment, it was observed that the shrinkage potential of MRF_60-40 was lower than the residual soil. However, cracks still occurred in the MRF_60-40 after several wetting and drying processes. Figure 6.22 shows the cracks observed in the MRF_60-40 layer after the drying process in Test CBM_MRF-GS_RF4.

Figure 6.22 Cracks observed after the drying process in Test CBM_MRF-GS_RF4

6.4.4 Concluding remarks

The infiltration tests in the capillary barrier model using a mixture of residual soil and fine sand (MRF_60-40) as the fine-grained layer (CBM_RS-GS) showed that the

capillary barrier model was able to store the infiltrating water in the fine-grained layer up to its water storage capacity without significant percolation.

As expected, by mixing the residual soil with fine sand, the recovery rate of water storage in MRF_60-40 was faster as compared to that of the residual soil. Lateral diversion flow was observed after rainfall and then followed by water removal by evaporation. However, the water balance and volumetric water content measurements indicated that water which flowed laterally in the fine-grained layer did not leave the capillary barrier model. Water was held in the lowest part of the capillary barrier model due to the low coefficient of permeability of the fine-grained layer. Moreover, desiccation cracks were still observed in the MRF_60-40 layer after several wetting and drying processes. Therefore MRF_60-40 is not recommended to be used as fine-grained layer due to its low water storage capacity, slow water storage recovery rate, and also the occurrence of desiccation cracks.

6.5 Geosynthetic Material as a Coarse-grained Layer in Capillary Barriers

6.5.1 Preliminary study

The use of geosynthetic material as a coarse-grained layer in a capillary barrier was investigated in this study. The geosynthetic material investigated in the experiment was Polyfelt Megadrain 2040 manufactured by Polyfelt Geosynthetics. Polyfelt Megadrain 2040 is a drainage product made of a matrix of polypropylene monofilaments sandwiched between two layers of polypropylene geotextile. It should be noted that water in the capillary barrier system is designed to flow above and within the geotextile and not in the monofilaments mat. Therefore, the water characteristic curve tests were conducted on the filter geotextile material. Drying and wetting water characteristic curves of the filter geotextile are presented in Figure 5.9.

Previous research works (Baker and Hillel, 1990; Stormont and Anderson, 1999; Tami, 2004) indicated that breakthrough into the coarse-grained layer will only occur if the matric suction in the coarse-grained layer reaches its water-entry value.

Therefore, it is preferred to have a coarse-grained layer that has a low water-entry value. The water-entry value, ψ_w , of the filter geotextile is 0.264 kPa, which is slightly lower than that of the gravelly sand used as the coarse-grained layer in CBM_RS-GS and CBM_MRF_GS (0.285 kPa).

6.5.2 Application of geosynthetic material in capillary barrier models

Two models were constructed using the geosynthetic material as the coarse-grained layer. A soil column model was constructed using 0.51 m thick fine sand overlying the geosynthetic material (SCM_FS-GE). The second model was a sloping capillary barrier model constructed using 0.2 m thick fine sand overlying the geosynthetic material (CBM_FS-GE).

The tests in SCM_FS-GE was initiated with a rapid drawdown test, which was conducted by lowering the water table from the top of the soil column ($z = 1.0\text{m}$) to the bottom of the soil column ($z = 0$). The rapid drawdown test was carried out to investigate the development of negative pore-water pressure during a drying process starting from the saturated condition. Figure 5.28 shows the development of pore-water pressure (PWP) head during the drawdown process in SCM_FS-GE. The PWP head profile of SCM_FS-GE obtained at the end of the drawdown test can be explained based on the water characteristic curves (WCC) and permeability functions of the materials. At the initial stage of drawdown test, volumetric water content and permeability of the materials were relatively high so that water could flow downward easily. With time, the volumetric water content decreased so that the permeability of the materials also decreased. Since the pore size of the geosynthetic material was much larger than the pore size of the fine sand, the volumetric water content of the geosynthetic material decreased faster than that of the fine sand. As the volumetric water content of the geosynthetic material approached its residual value, the water phase became discontinuous so that the permeability of the geosynthetic material became very low. At this point, it was difficult for water from the geosynthetic material and the fine sand above it to flow downwards. Based on the WCC (Figure 5.9), the residual matric suction value of the geosynthetic material was about 3 kPa (equal to PWP head of around -0.3 m).

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This value was comparable with the PWP head measured by tensiometer in the fine sand layer just above the geosynthetic material. From this point upwards, a hydrostatic PWP head profile was developed up to a value of -0.5 m, followed by an almost vertical PWP head profile up to the top of the model. This type of profile was developed due to the fact that the residual matric suction of the fine sand was around 5 kPa (equal to PWP head of -0.5 m), so that the minimum PWP head developed in the fine sand was also around -0.5 m. The observations on the minimum PWP that developed in the capillary barrier models were consistent with the previous study on layered soil columns by Choo and Yanful (2000). The study showed that PWP in soil columns stabilized at a value near the soil's residual matric suction. In the fine sand layer below the geosynthetic material, the minimum PWP that were measured was around -0.45 m. This value corresponded to a hydrostatic profile with water table at the bottom of the soil column ($z = 0$).

Another soil column model was constructed using a fine sand layer overlying a gravelly sand layer (SCM_FS-GS). After drawdown a process, the geosynthetic material (Figure 5.28) developed a smaller PWP break as compared to the gravelly sand layer (Figure 5.24). The PWP head in the fine sand above the geosynthetic material was lower than that above the gravelly sand layer. This condition was favorable because the unsaturated permeability of the soil would be lower at a low PWP head (high matric suction). Hence, it would be more difficult for water to flow in the fine sand layer above the geosynthetic material than in the fine sand above the gravelly sand layer during rainfalls after the drawdown process. Results of rainfall tests on the soil column models indicated that the above statement is plausible. Three similar rainfall tests were applied to each soil column model (Table 4.3). Figures 5.25, 5.26, and 5.27 show the PWP head profiles during tests SCM_FS-GS_RF1, SCM_FS-GS_RF2, and SCM_FS-GS_RF3, respectively. The PWP head profiles of other tests with similar rainfall intensities and durations on SCM_FS-GE (tests SCM_FS-GE_RF1, SCM_FS-GE_RF2, and SCM_FS-GE_RF3) are presented in Figures 5.29, 5.30, and 5.31. It can be seen that generally there was no change in the pore-water pressure measured in the fine sand below the geosynthetic layer. However, in the fine sand below the gravelly sand layer, the pore-water pressure increased during tests SCM_FS-GS_RF1 and SCM_FS-GS_RF2.

This is an indication that water did not flow through the geosynthetic material in SCM_FS-GE but flowed through the gravelly sand layer in SCM_FS-GS.

The second model constructed using the geosynthetic material was a sloping capillary barrier model which consisted of a 0.2 m thick fine sand overlying the geosynthetic material (CBM_FS-GE). The performance of CBM_FS-GE was studied by conducting several rainfall tests as summarized in Table 4.4.

One of the criteria to determine the performance of capillary barrier is the water storage capacity of the fine-grained layer at the onset of percolation into the coarse-grained layer. Based on the water balances, percolation through the upper geotextile layer occurred in Tests CBM_FS-GE_RF3, CBM_FS-GE_RF4, and CBM_FS-GE_RF6. The percolations were indicated by water flow in the geonet drainage layer. The water balances (Figures 5.60, 5.62, and 5.66) show that the onset of drainage from the geonet was at elapsed times of 50 hr, 25.7 hr, and 1 hr for tests CBM_FS-GE_RF3, CBM_FS-GE_RF4, and CBM_FS-GE_RF6, respectively. The amount water stored in the fine-grained layer at the onset of percolation is summarized in Table 6.4.

Table 6.4 Water storage capacity of the fine-grained layer in CBM_FS-GE

Name of Tests	T (hr)	S (L)
CBM_FS-GE_RF3	50	50.5
CBM_FS-GE_RF4	25.7	52.6
CBM_FS-GE_RF6	1	54.0
Note: T = Elapsed time at the onset of drainage from the geonet S = Amount of water stored in the fine-grained layer when drainage from the geonet occurred		

It can be seen from Table 6.4 that percolation into the coarse-grained layer (geosynthetic material) occurred when the amount of water stored in the fine-grained layer is above 50.5 L.

As discussed in Section 6.3.1, Stormont and Morris and (1998) proposed a method to calculate the water storage capacity of capillary barriers. This method was also used to calculate the water storage capacity of CBM_FS-GE. Figure 6.23 illustrates the calculation of water storage capacity of CBM_FS-GE. Based on this method, the water storage capacity of CBM_FS-GE is 51 L.

Figure 6.23 Matrix suction and volumetric water content profile to estimate the storage capacity of CBM_FS-GE

The value (51 L) calculated using the method proposed by Stormont and Morris (1998) is very close to the experimental results (50.5 L). Therefore, this method could be used to estimate the water storage capacity of a capillary barrier system.

Based on the water balances of the tests in CBM_FS-GE, the highest amount of water stored in the fine-grained layer occurred in Test CBM_FS-GE_RF6 (65 L). As shown in Table 6.4, percolation into the geosynthetic layer occurred in Test CBM_FS-GE_RF6 when the amount of water in the fine-grained layer was 54 L. However, water balance measurement indicated that there was no percolation into the fine sand layer below the geosynthetic layer. The water balance was also supported by the volumetric water content measurement during Test CBM_FS-GE_RF6 (Figure 6.24). There were no volumetric water content changes observed

in the fine sand layer below the geosynthetic. The results indicated that water which percolated through the upper geotextile layer was drained through the geonet drainage layer. In this case, drainage needs to be provided to drain water from the geonet drainage layer.

Figure 6.24 Volumetric water content changes in Test CBM_FS-GE_RF6

6.5.3 Numerical analyses on the capillary barrier using geosynthetic material

Numerical analyses were conducted to simulate the tests on the soil column model (SCM_FS-GE) and capillary barrier model using geosynthetic material as the coarse-grained layer (CBM_FS-GE). Figure 5.70 shows the comparison of pore-water pressure head profiles during a drawdown process in SCM_FS-GE. It can be seen from Figure 5.70 that the numerical analysis was able to simulate the drawdown process closely. The numerical results from SCM_FS-GE and CBM_FS-GE agree reasonably well with the experimental data, especially during wetting processes (Figure 5.71 and 5.78). During drying processes, the pore-water pressure heads obtained from the numerical simulation results moved faster towards the hydrostatic condition as compared to those obtained from the experiments (Figures 5.70 and 5.72). This behaviour was also observed in a previous numerical study by Choo and Yanful (2000). It was due to the fact that the coefficient of

permeability of the fine sand is low enough to sustain pressure over long periods of time as the residual matric suction of the soil reached. However, the permeability function of the fine sand used in the numerical simulation did not account this phenomenon. Therefore, the pore-water pressure head from the numerical analysis keep decreasing while the experimental data already stabilized.

In the numerical analyses of sloping capillary barrier model, the direction of water flow during rainfall could be obtained based on the hydraulic gradients in the soil. Figure 6.25 shows the total head contour and the direction of water flow in the fine-grained layer (fine sand) at an elapsed time of 6 hours in test CBM_FS-GE_RF2. It can be seen that water flowed laterally in the fine sand layer above the geosynthetic material. This behaviour was also observed in the experiment as the lateral diversion flow.

Figure 6.25 Total head contour and direction of water flow in the fine-grained layer at an elapsed time of 6 hours in test CBM_FS-GE_RF2

6.5.4 Concluding remarks

In the soil column model, the geosynthetic material developed a smaller PWP break as compared to the gravelly sand layer. The PWP head in the fine sand

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above the geosynthetic material was lower than that above the gravelly sand layer. As a result, the unsaturated permeability of the fine sand above the geosynthetic material was lower than that of the fine sand the above gravelly sand. Therefore, it would be more difficult for water to infiltrate the fine sand layer above the geosynthetic material than the fine sand layer above the gravelly sand layer during rainfalls.

As indicated by the results on the sloping capillary barrier model, the geosynthetic material provided double protection so that water could flow above the upper geotextile layer as a lateral drainage flow under unsaturated condition and also inside the geonet drainage layer if percolation through the upper geotextile layer occurred. The numerical analysis result also shows that lateral diversion flow occurred in the fine sand layer.

Based on the experimental and numerical results of the soil column model and the sloping capillary barrier model using fine sand as the fine-grained layer and geosynthetic material (Polyfelt Megadrain 2040) as the coarse-grained layer, it can be concluded that the geosynthetic material used in this study performs effectively as a coarse-grained layer in capillary barrier system.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

7.1.1 Feasibility of using residual soil and mixture of residual soil and fine sand as fine-grained layers of capillary barriers

Some conclusions from the study on the residual soil and mixture of residual soil and fine sand are listed as follows:

1. The experimental and numerical analyses results indicated that the capillary barrier model using residual soil as the fine-grained layer was able to store the infiltrating water in the fine-grained layer up to its water storage capacity without significant percolation. The evaporative flux applied to the capillary barrier model could remove water from the residual soil layer at a faster rate. For illustration, the rate of water storage changes in Test CBM_RS-GS_RF1 (evaporation under laboratory ambient conditions) was 0.017 L/hr, while in Test CBM_RS-GS_RF4 (accelerated evaporation using lamp and fan), the rate of water storage changes was 0.163 L/hr. However, these rates were still much lower than the rate of water storage changes in a fine sand layer due to lateral diversion flow (4 L/hr). The applied evaporative flux to CBM_RS-GS resulted in excessive desiccation cracks. Therefore, due to its slow rate of water storage recovery and also the excessive desiccation cracks during drying process, the local residual soil of Bukit Timah Granite investigated in this study is not recommended to be used as a fine-grained layer in a capillary barrier system.
2. The evaporation and shrinkage test results indicated that mixing the residual soil with fine sand could reduce the shrinkage potential and improve the hydraulic properties of the soil. The volumetric strain of the

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residual soil was reduced from 23% to 18% by mixing the residual soil with fine sand at a composition of 60% residual soil and 40% fine sand by dry mass. The saturated coefficient of permeability of the residual soil increased from 8.57×10^{-8} m/s to 2.71×10^{-7} m/s by mixing the residual soil with fine sand at a composition of 60% residual soil and 40% fine sand by dry mass. The mixture of residual soil with fine sand at a composition of 60% residual soil and 40% fine sand by dry mass (MRF_60-40) was used as the fine-grained layer in CBM_MRF-GS. The measurements of pore-water pressures and volumetric water contents indicated that lateral diversion flow occurred in CBM_MRF-GS. However, the water balance and volumetric water content measurements suggested that water which flowed laterally in the fine-grained layer was held in the lowest part of the capillary barrier model due to the low coefficient of permeability of the mixture. Therefore, the MRF_60-40 is not recommended to be used as a fine-grained layer due to its low water storage capacity, slow rate of water storage recovery, and also the occurrence of desiccation cracks.

7.1.2 Feasibility of using geosynthetic material as coarse-grained layer of capillary barriers

From the results of experiments and numerical analyses of capillary barrier model using fine sand as the fine-grained layer and geosynthetic material (Polyfelt Megadrain 2040) as the coarse-grained layer it can be concluded that the geosynthetic material used in this study performed satisfactorily as a coarse-grained layer in the capillary barrier system. The geosynthetic material provides protection so that water did not percolate into the original soil protected by the capillary barrier system. The experimental results showed that water could flow above the geosynthetic material as lateral drainage under unsaturated condition and also inside the geonet drainage layer if percolation through the upper geotextile layer occurred.

In the soil column model tests, the geosynthetic material developed a smaller pore-water pressure break as compared to that in the gravelly sand layer after a

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drawdown process. The pore-water pressure head in the fine sand above the geosynthetic material was lower than that above the gravelly sand layer. This condition was favorable because unsaturated permeability of the soil would be lower at a low pore-water pressure head (high matric suction). Hence, it would be more difficult for water to flow in the fine sand layer above the geosynthetic material than in the fine sand layer above the gravelly sand layer during rainfalls.

7.1.3 Tests and calculation methods adopted in this study

Some conclusions on the test and calculation methods adopted in this study are listed as follows:

1. The capillary rise test adopted in this study could produce consistent results of drying and wetting water characteristic curves of the geotextile. Several tests with different durations should be conducted to confirm the equilibrium condition of the geotextile.
2. Instantaneous profile method could be used to determine permeability functions of the soils used in the capillary barrier models accurately. Results of numerical simulations using permeability functions obtained from the instantaneous profile method were in good agreement with the experimental data.
3. The evaporation and shrinkage test adopted in this study could be used to assess the shrinkage characteristics of the materials proposed to be used as the fine-grained layer in capillary barrier system. The results of evaporation and shrinkage tests under different evaporation rates indicated that shrinkage curve of a soil is not dependent on the evaporation rate. Shrinkage curve is a function of water content or matric suction in the soil.
4. The numerical model adopted in this study could simulate the laboratory experiment closely, especially during the wetting process. Some discrepancies between experimental and numerical analyses results during the drying process were observed due to the limitation of permeability function used in the numerical analyses. However, in general, the numerical model was able to simulate the drying process reasonably well.

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5. The method to calculate water storage capacity of capillary barrier as proposed by Stormont and Morris (1998) was found to be sufficiently accurate. For example, based on this method, the water storage capacity of CBM_FS-GE was 51.03 L. The experimental results indicated that percolation occurred when the water stored in the fine sand was above 50.5 L, which was very close to the computed value.

7.2 Recommendations

Further studies recommended in the field of capillary barrier as a slope protection cover are listed as follows:

1. The design and construction aspects, such as the dimension of the capillary barrier, determination of surface drainage location, and the stability during and after construction requires further investigations.
2. A full-scale field experiment of capillary barrier system on a slope with comprehensive instrumentation and monitoring systems could be carried out in order to study the performance of the capillary barrier system in the field as compared to the performance observed in the laboratory model.

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APPENDIX A

Properties of Polyfelt Megadrain 2040 (Polyfelt Geosynthetics)

Properties of the Drainage Net

1. in-plane flow capacity, q
i = 1	20 kPa	:	3.30	l/m.s
	50 kPa	:	1.40	l/m.s
i = 0.1	20 kPa	:	0.90	l/m.s
	50 kPa	:	0.70	l/m.s
i = 0.04	20 kPa	:	0.50	l/m.s
	50 kPa	:	0.15	l/m.s
2. Thickness
2 kPa	:	20	mm
3. Tensile strength
	:	20	kN/m

Properties of the Filter Layer

1. Cone drop test
	:	30	mm
2. Tensile strength
	:	9.5	kN/m
3. CBR puncture resistance
	:	1600	N
4. Opening Size O₉₀
	:	110	µm
5. Water permeability vertical
	:	100	l/m ² .s

APPENDIX B

Boundary conditions at the bottom of the soil column model used in the numerical simulation of drawdown tests:

1. SCM_FS-GS

Figure B.1 Boundary condition at the bottom of the soil column model used for the numerical simulation of test SCM_FS-GS_DD

2. SCM_FS-GE

Figure B.2 Boundary condition at the bottom of the soil column model used for the numerical simulation of test SCM_FS-GE_DD

APPENDIX C

The following photos show the trial application of capillary barrier system as slope cover against rainfall-induced slope failure. The capillary barrier system was constructed using fine sand as the fine-grained layer and a geosynthetic material as the coarse-grained layer. The fine sand was contained using geocell and placed above the geosynthetic material. Top soil was placed above the fine sand layer and grass was grown on the surface.

Figure C.1 Filling the geocell with fine sand (the fine-grained layer)

Figure C.2 Details of geocell (with holes to allow lateral diversion)

Figure C.3 Sub-surface and surface drainage provided in the capillary barrier system

Figure C.4 Filling the geocell with top soil (above the fine sand)

Figure C.5 Placement of grass

Figure C.6 Final condition with tensiometers installed (inside the box; for slope monitoring purpose)